Numerical Modelling of River Rehabilitation Schemes

by Neil Swindale, BEng(Hons)

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Corrigendum

All values shown on figures, plans etc are in SI units. Distances are in metres, pressures in Newtons per metre squared, shear stresses in Newtons per metre squared and velocities in metres per second.

The flow direction in all cases is vertically upwards on the page, unless indicated otherwise by flow direction arrows (as in pages 120-122).

All layout plans are orientated with north upwards on the page.

Abstract

This thesis is based on the application of hydraulic modelling techniques to the study of river rehabilitation schemes. River channelization and rehabilitation techniques are reviewed and the restoration of the River Idle is detailed. The rehabilitation of the Idle, consisting principally of the installation of a number of flow deflectors, forms the basis of the modelling work carried out.

Open channel modelling techniques are reviewed and the packages ISIS, HEC-RAS, SSIIM and CFX are applied to the River Idle. Results from SSIIM (two dimensional) and CFX (three dimensional) are validated against site measured velocities. SSIIM predicted velocities calibrate poorly against site data whilst CFX results are considerably more encouraging. Reasons for the increased accuracy of the three dimensional results are discussed.

The effect of the installation of the flow deflectors on aquatic habitat is simulated using the techniques underlying the Instream Flow Incremental Methodology (IFIM). The results from the one dimensional model ISIS and the three dimensional package CFX are used to make available habitat predictions. Results indicate an improvement in habitat for adult and spawning chub but a worsening of habitat for roach fry. However, habitat for roach fry can be expected to improve with time as the geomorphology of the river responds to the installation of the deflectors.

The results from the habitat modelling exercise also indicate significant discrepancies between the results obtained by applying the one and three dimensional models. Greater improvements in habitat are indicated in the results from the three dimensional modelling approach. This can be attributed to a number of factors but most significantly the fact that the three dimensional model, in solving two further momentum balance equations, accurately simulates a plume of higher velocity which is produced by the narrowing of the channel width at the deflector. This plume of higher velocity is propagated downstream for some distance beyond the deflector and is associated with improved habitat suitability in the case of adult and spawning chub.

The effect of the deflectors on the movement of sediments in the Idle is simulated using ISIS Sediment, a module of the ISIS package, and SHEAR. SHEAR is a FORTRAN program, written for this thesis, which calculates bed shear stresses from the vertical velocity distribution predicted by CFX. The predicted bed shear stresses are compared with a critical shear stress for erosion which is calculated from the Shields criteria. Deposition areas can be implied from zones of reduced bed shear stress. Thus, SHEAR is able to describe the spatial detail of erosion and deposition, for any given sediment particle size, at a specific discharge. Results from ISIS Sediment and SHEAR are compared qualitatively with site measurements of bed erosion that has taken place at a single deflector site. Results indicate that the programs have successfully reproduced the major features of the movement of sediments observed on site. These consist of the erosion of a scour pool adjacent to the deflector tip and deposition in the lee of the deflector leading to the development of a bank of sediment.

Overall, significant benefits are indicated in a three dimensional approach over the more traditional one dimensional models. These are evident in both improved calibration with site measured velocities, better available habitat prediction and the ability to describe the spatial detail of erosion and deposition.

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Finally, I would like to thank my parents for their continued support.

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Notation

B	Channel width
b_e	Bed elevation
c_T	Sediment concentration (ppm)
D_m	Hydraulic Mean Depth
D	Diameter of sediment grains
E	Loglayer constant
g	Acceleration due to gravity
H	Depth at a point
k	Turbulent kinetic energy
K	Mean kinetic energy
k_s	Effective grain roughness
l_m	Mean flow length scale
p	Pressure
P	Stream power
Q	Discharge
q	Discharge per unit width
q_s	Sediment transport rate (m^3/s)
R	Hydraulic radius
S_m	Momentum source term
S	Water surface slope
S_0	Bed slope
S_f	Slope of the total energy line
t	Time
U	Steady mean value of velocity in the x direction
u	Instantaneous Velocity in the x direction

u_{max} or U_{max}	The largest velocity in a distribution
u'	Fluctuating component of velocity in the x direction
u^+	Scaled dimensionless velocity in the near wall region
u_*	Shear velocity
v	Instantaneous velocity in the vertical plane (y direction)
V	Steady mean value of velocity in the vertical plane(y direction)
v'	Fluctuating component of velocity in the vertical plane (y direction)
W	Steady mean value of velocity in the z direction (perpendicular to U)
w	Instantaneous velocity in the z direction (perpendicular to u)
w'	Fluctuating component of velocity in the z direction (perpendicular to u')
x	Distance in direction of u component of velocity
y	Distance in the vertical plane
y^+	Scaled dimensionless distance from the boundary in the near wall region
z	Distance in direction of w component of velocity
λ	Bed Porosity
ν	Kinematic viscosity
μ	Absolute viscosity
τ	Shear stress
τ_w	Shear stress at the wall/boundary
τ_{CR}	Critical shear stress for bed erosion to take place
ρ	Fluid density
ρ_s	Sediment density
γ_s	Specific weight of sediment
γ_f	Specific weight of the fluid
ϕ	Friction angle of sediment
ν_t	Kinematic eddy viscosity
μ_t	Turbulent eddy viscosity
ϑ	Velocity length scale of turbulence
ι	Length scale of turbulence
C_μ	Dimensionless constant used in the k-ε model
σ_k	Dimensionless constant used in the k-ε model

σ_ε	Dimensionless constant used in the *k*-ε model
C_k	Dimensionless constant used in the *k*-ε model
$C_{z\varepsilon}$	Dimensionless constant used in the *k*-ε model
κ	Von Karman constant
σ	Height of the boundary layer
ε	Rate of viscous dissipation

Chapter 1

Introduction

The purpose of this thesis is to examine what useful data can be obtained from current state of the art open channel modelling packages, with regard to the design and analysis of river rehabilitation schemes. The thesis focuses on the interpretation of the data that can be obtained with regard to different disciplines, but most specifically in its application to ecohydraulics. Thus, this is fundamentally a thesis about the hydroinformatic possibilities in data obtained from different packages, as opposed to an examination of numerical modelling techniques. However, some consideration will be given to the strengths and difficulties of various modelling approaches.

The work outlined in the ensuing chapters of this thesis relates specifically to the modelling of the rehabilitation works carried out on the River Idle. However, the methods used are intended to be applicable to rehabilitation schemes in general. Indeed, it is considered that the approach which is being described here can contribute to an improved means for designing river rehabilitation schemes. It should also be pointed out that all of the modelling work detailed in the subsequent chapters of this thesis was completed after the rehabilitation measures had been installed.

In the past many of the rivers in the U.K. have been channelized. A brief history of channelization, the reasons for carrying it out and its detrimental effects are included in chapter 2. In order to mitigate against the harmful effects that channelization has on aquatic species and the amenity value of watercourses, there has been a recent trend towards rehabilitating and restoring channelized rivers. Rehabilitation techniques in general are also discussed in chapter 2, together with the rehabilitation of the River Idle and the problems specific to that design.

Rehabilitating rivers in order to increase the number of aquatic species present is not straightforward. The habitat requirements of individual fish species and communities are still not fully understood. As a consequence many rehabilitation projects are founded on 'best guess' professional judgement, based on the belief that creating wildlife habitats is an art and not a science (Brookes et al, 1998). A quote from a recent meeting at the Institute of Civil Engineers on eco-hydraulics highlights this point:

'Too many restorations in the past have been carried out with no quantitative prediction of the impact on the flow or on the environmental enhancements that will be achieved. Increasingly suitable quantitative methods are becoming available and it is a challenge for the research community to provide methods, in collaboration with the ecologists, which will predict the true impact of restoration. It is time that we replaced guesswork with accurate prediction.' (Bettess, 1997)

In this thesis, a habitat model is used to assess the impacts of the rehabilitation work on fish species. The use of a model has the benefit that it provides a quantitative numerical system, which can be assessed objectively. The model used should provide common ground and, hopefully, reduce the scope for debate. A number of habitat models are available, many of which are based on broad assumptions. As a result they have encountered widespread criticism. This is not helped by the fact that the models are limited by the current state of knowledge of the requirements of fish habitat. Habitat modelling, its current limitations and some of the programs available are reviewed in chapter 5. The current state of knowledge can be summarised to an extent by the following quote:

"Predictive capacity (of habitat models) is questionable. We can explain observed phenomenon but not necessarily predict how species will react to imposed changes." (Clifford, 1998)

Habitat modelling has a long way to progress to become a completely reliable predictive tool. However, the existing models are at least partially successful in predicting habitat change and provide the starting point from which more detailed techniques can be developed.

The modelling of aquatic habitat on the River Idle carried out for this thesis uses the concepts that underly the Instream Flow Incremental Methodology (IFIM). This is the most popular approach currently available. Here the habitat requirements of individual 'target' species of fish are described by 'preference curves', which relate habitat suitability to flow depth, velocity and the bed substrate. The IFIM technique is usually only applied to one dimensional modelling. One dimensional models generally need extensive calibration against site velocities, and cannot be relied upon to predict velocities across the channel width after any alterations to the channel. The opportunities presented by the development of two and three dimensional models have implications for habitat modelling:

"It is a challenge to the habitat modelling community to decide what is the required level of accuracy (1D, 2D, or 3D) for assessing habitat improvements." (Bettess, 1998)

In chapter 5 the IFIM technique is applied to one and three dimensional models to make an assessment of the effect that the rehabilitation measures on the River Idle will have on aquatic habitat. A number of FORTRAN programs were written by the author in order to apply the IFIM approach to the different sets of results. Results from the analysis are presented in the form of colour plots of available habitat and graphs of Weighted Usable Area (WUA) of habitat. Using two and three dimensional flow models at the design stage should allow accurate prediction of the flow patterns that will arise after rehabilitation measures are constructed. Thus, using the techniques outlined, habitat improvements arising from alternative design proposals should be quantifiable at the feasibility/design stage.

Rehabilitating any river with the sole aim of improving wildlife habitats presents certain problems. Firstly, when the river was channelized it will have been carried out with a specific purpose in mind. It is important that the rehabilitation design does not compromise this purpose. The purpose of channelzation was, in many cases, improvement of flood defence. Channelization via straightening, widening and deepening increases conveyance and correspondingly reduces flow depths. Re-introducing meanders, and installing features which promote recovery, effectively increases the resistance to flow. As a result, it is necessary to test rehabilitation design proposals against their effect on flood peak levels. This is carried out in chapter 4 of this thesis for the case of the River Idle. The one dimensional open channel flow models ISIS and HEC-RAS are used for this purpose, and the results from the two packages are compared. At the same time, more detail of the effect of the rehabilitation on flow patterns is obtained using the two dimensional model SSIIM and the three dimensional model CFX. The predicted velocities from both SSIIM and CFX are also validated against site measurements. One dimensional modelling in general and the packages ISIS and HEC-RAS are discussed in chapter 3. Two dimensional modelling using SSIIM, and three dimensional modelling using CFX are discussed in the same chapter.

The importance of ensuring that habitat improvements do not compromise flooding risk is highlighted by a recent publication which reports on the extensive flooding that took place in 1998. This report highlights the fact that some of the flooding may have been due to the construction of rehabilitation schemes. This is summarised by the following quote:

'The (Environment) Agency should consider whether enhancing the nature conservation value of watercourses without compensating flood defence action is increasing upstream urban flood risks' (Independent review team, 1998).

Probably the most important factor in designing rehabilitation measures is to ensure the design does not create excessive instability in the watercourse. Designs that are not appropriate for the intended river, are liable to either be eroded or covered in deposited sediment. The following quote describes this well:

'In many instances, the construction of inappropriate habitat features are unsustainable and, moreover, compromise the flood capacity and sediment transport

regime of the river. The requirement, therefore, is to develop natural channel design procedures. By designing with nature, appropriate and sustainable habitat features can be created which are in harmony with the physical processes controlling the functioning and form of river systems. Thereby, the maximum potential biodiversity of the river can be both realised and sustained.' (Hey, 1997a)

The main point here is that the temptation provided by the recent rapid growth in popularity of habitat modelling is to try to engineer habitats for species which would not naturally be found in the watercourse under consideration. Inevitably this type of approach leads to the creation of unnatural features within the river that can be rapidly destroyed. The potential pitfalls of this type of approach were highlighted at a recent meeting:

"Who decides if you design a river specifically to improve habitat for salmonids?"

"The answer is God decides" (Hey, 1998)

The answer suggests that the actions of erosion and deposition will destroy any habitat features constructed that are not in harmony with the natural processes in the river. This phenomenon has already been observed at some of the demonstration sites for river rehabilitation set up in the U.K. (Hey, 1997b). It is far better to attempt to create a condition within the river that represents a stable situation. In most cases this condition will be described by that which closely matches how the river would appear if it had never been channelized. In this case, features will be sustainable and the river should be colonised by aquatic species which would naturally be resident in the habitats that are provided.

So, the question as far as river rehabilitation is concerned becomes how to improve aquatic habitat, without significantly increasing the risk of flooding, and at the same time create a design which is in harmony with the natural processes in the river. The analysis presented suggests that input from an ecologist, geomorphologist and civil engineer is required. The role of the ecologist is to advise on habitat requirements in the context of the reach of river under consideration. The roles of the engineer and geomorphologist are complimentary as described by Brookes:

'Whereas the engineer is concerned with the design, siting and construction of particular structures, the geomorphic approach entails background knowledge of the function of structures, together with an understanding of the local and regional effects. The participation of the geomorphologist in river engineering should therefore be sought at the planning stage and not as an afterthought once problems have arisen. Such participation will avoid failure of structures, unexpected extra costs due to maintenance and loss of time.' (Brookes, 1988)

This thesis is written from an engineering perspective, however the importance of the contribution of the geomorphologist to ensuring stability of a rehabilitation design is appreciated and is commented on in detail in chapters 2 and 6. In essence, the geomorhpologist can be seen to be more concerned with the larger picture of the river and its evolution over relatively large time scales. The civil engineer tends to be more focused on the specific detail of designs and their effect in more localised areas, and within a shorter time frame. Inputs from both professions are required to ensure a successful design.

In the analysis presented here, the effect of a rehabilitation design on channel stability is assessed using one and three dimensional modelling as shown in chapter 6. Some background to sediment transport modelling is given in the same chapter. The one dimensional modelling is carried out using ISIS Sediment, a module of the ISIS package. The three dimensional modelling is carried out by assessing the effect of the pattern of velocities predicted by CFX on bed shear stresses. A FORTRAN program was written by the author to carry out this analysis. Results are presented in the form of colour plots of bed shear stress and eroded areas of bed. To fully appreciate the significance of both of the set of results generated, they would have to be considered within the context of overall channel stability,as assessed by a geomorphologist.

The research work presented here takes advantage of the possibilities presented by two and three dimensional modelling for the prediction of the three key areas of: flooding (and the effect on flow patterns), aquatic habitat and the movement of sediments (hence channel stability). This is a novel approach as the vast majority of the work carried out in industry is still wholly dependant on a one dimensional approach. The validation of the predictions from SSIIM and CFX against site measured velocities presented in chapter 4 gives an indication of the potential accuracy that can be obtained with two and three dimensional modelling packages. The results from the habitat modelling in chapter 5 show how the increased accuracy of the flow predictions from three dimensional modelling can be transferred to more reliable and sound predictions of habitat change following rehabilitation. A one dimensional model is shown to be unable to predict velocities following rehabilitation with sufficient accuracy, as it only solves a momentum equation in one direction. Two and three dimensional models solve for the flow pattern more accurately by solving additional momentum equations. The results in chapter 5 show how the one dimensional habitat modelling results under predict the habitat gain from installing flow deflectors. The more reliable predictions from the three dimensional model show considerably larger benefits, in terms of habitat improvement, arising from the rehabilitation scheme.

The sediment modelling work presented in chapter 6 shows how the vertical velocity distribution, predicted by the three dimensional modelling package CFX, can be used to make predictions of bed shear stress. These bed shear stresses can be used in turn to predict where erosion will take place in great spatial detail. This technology holds considerable potential benefits in assessing the stability of rehabilitation designs.

The central theme of this thesis is that the benefits in using two and three dimensional hydraulic models, for assessing the entire effect of river rehabilitation schemes, are substantial. The increased accuracy that can be obtained in assessing habitat improvements is demonstrated, together with the greater appreciation of likely patterns of erosion and deposition (and hence channel stability). At the present time, the potential possibilities in two and three dimensional modelling are restricted by the lack of a package tailored to the needs of the river modelling community. Research needs to be directed to the development of a three dimensional package which contains all the essential features that river modellers require. Once such a package becomes available, river modellers should be actively encouraged to take advantage of the new technology so that the potential benefits described here can contribute to the design of better rehabilitation schemes.

The consideration of the three key elements in any river rehabilitation are presented here as part of an integrated modelling strategy, such that the substantial effect of a scheme can be determined at the design stage. Again this is different to the design procedure followed in industry where generally only one or two of the relevant factors are considered (often stability considerations are omitted as has been discussed above). Essentially an approach is being proscribed that involves modelling in much more detail, taking *all* of the relevant factors into account and obtaining input from specialists in each of the various disciplines at the design stage of a rehabilitation. As a result a more informed choice between alternative rehabilitation proposals can be made.

Chapter 2

Review of River Restoration

2.1. Introduction

This chapter begins with a discussion of river channelization. There follows an analysis of the harmful effects that channelization can produce. Leaving aside the poor aesthetics, which is the result of most channelization schemes, the negative results can be broadly categorised into physical and biological.

All rivers tend towards a condition of equilibrium whereby the geometry and planform have arisen from, and are in harmony with, the action of the discharge and sediment hydrographs on the soil or rock that is present. As an example of this, rivers meander because the gradient they require to transport the sediment load is less than the valley slope. Channelization disrupts the equilibrium condition. As a result, the physical effect of channelization is that the river will attempt to return to a stable condition. This involves erosion or deposition and these processes can destroy the engineering works that have been constructed. Channelization also has a detrimental effect on the aquatic environment. This is due, to a considerable amount, to the fact that there is a lack of habitat diversity in a channelized reach.

Some rivers will recover naturally after being channelized, but others will not. The reasons for this are explored here. In order to improve the natural habitat, and aesthetic quality, many rivers have been reengineered following channelization. This can either involve rehabilitation, where the channelization design is left in place but mitigating features are installed, or the complete restoration of the river to its pre channelization state. In either case, consideration must be given to whether the new scheme will be stable and in sympathy with the natural processes within the river. Current thinking on the subject is reviewed including the regime and rational equations, which can be used to help to predict stable channel geometry and planform.

Unfortunately, there are potential drawbacks in the use of both the rational and regime equations. These are detailed later in this chapter. As a result, it is necessary to devise alternative means to assess the effect of proposed designs, in detail, on channel stability. The sediment modelling techniques employed in chapter 6 are a step in this direction. In conjunction with a morphological survey, they can be used to gain a good appreciation of the likely effect of any changes. This matter, as well as a general discussion of the roles of the civil engineer and the geomorphologist in ensuring stability of a rehabilitation design, is discussed in detail in chapter 6.

A number of techniques can be used for rehabilitation. The work carried out in this thesis focuses on the installation of flow deflectors on the River Idle in North Nottinghamshire. The detail of this rehabilitation scheme is given at the end of the chapter.

2.2. Channelization

River channelization involves the direct modification of river channels. It includes all the activities associated with altering a river channel for the purposes of: flood control, drainage improvement, navigation, control of bank erosion and highway construction. The main methods of channelization include: widening, deepening, straightening, construction of flood levees, bank stabilisation and the removal of trees and weeds (clearing and snagging). Some more detail is given below on the methods of channelization.

The aim of widening and deepening (resectioning) a channel is to increase the discharge that can be passed without the water elevation exceeding the height of the banks and flooding taking place. Typically, for flood control purposes, the size of the modified channel will be determined by the flood discharge which is to be contained. Trapezoidal cross sections are common as they provide relatively stable bank slopes. Commonly, channel resectioning is accompanied by regrading of the bed. Regrading can be undertaken to improve water levels for storm-water outfalls, or to improve agricultural drainage.

Channel realignment or straightening is usually carried out to improve a watercourse's ability to pass floods. A straighter channel offers reduced flow resistance, so flood waves pass more quickly. A number of procedures can be used to alter a channels alignment ranging from careful use of dredging, to the excavation of cutoffs to remove meanders.

Flood levees (also known as embankments, bunds or stopbanks) are one of the oldest forms of flood protection. They are constructed on top of the channel banks to increase a watercourse's capacity so that relatively high discharges remain within the channel. Like resectioning, levees are constructed to contain a design discharge. In some cases levees are constructed to a high level on both banks such that the entire floodplain is protected. In other cases, the levees can be places on the floodplain so that some flooding can take place. Trapezoidal levees are the most common, and the top of the levee should be wide enough to permit maintenance traffic.

Bank stabilization is carried out to protect against erosion and bank slips. One method of stabilization is the use of revetments. These consist of covering the bank with a protective layer of material. Commonly concrete, stone or a plastic sheet is used. Permeable revetments include willow pilings and gabions. Gabions are widely used and consist of rock filled wire cages.

Fully lined channels are usually limited to urban areas where other types of channelization cannot be employed due to the limit on space. Again their size is determined based on a design discharge. Fully lined channels are often rectangular in cross section, and are constructed from concrete or some form of sheet piling. No vegetation can grow in these channels, and no erosion can take place. They should be designed such that the gradient is steep enough to prevent any deposition of sediment.

Maintenance of channels, in the form of dredging and clearing and snagging, is necessary to keep the roughness sufficiently low so that the risk of flooding is not greatly increased. Dredging can consist of breaking up the material on a channel bed and allowing the current to take it away, or the excavation and removal of a substantial amount of material from the bed. The growth of weeds in a channel can physically reduce capacity as well as affecting the roughness. A number of methods are employed to remove excess growth. Clearing and snagging also includes the removal of fallen trees and debris jams.

2.2.1 History of Channelization

In the UK much channelization has taken place, principally for the purposes of flood alleviation, agricultural drainage or to make rivers more navigable. Brookes (1988) states that Lamplugh has found man-made alterations to the natural drainage system dating from the earliest cultivation. Changes to major rivers, for agricultural or navigation purposes, can be detected from the late 19th century. Often rivers were deepened and stabilised. In addition, bridges were constructed and weirs were put in place to control the channel gradient.

The history of channelization schemes, for the purpose of improving navigation, dates to the 14th and 15th century. During this time extensive silting of rivers is thought to have taken place, and as a result a number of acts were passed to aid navigation. From 1600 to 1750 many widening and deepening schemes were implemented following a substantial growth in population. Also in the 17th century many cutoff schemes were built to shorten the length of a river. Much of these early attempts at channelization required constant maintenance as the river tried to restore itself to equilibrium by increasing bed or bank erosion, or through the deposition of excess amounts of sediment. Brookes refers to Priestly who states that the improvement works on the Hampshire Avon between Christchurch and Salisbury were destroyed by a flood shortly after their construction during the 1660s. As a result later works were more substantial. After 1760 schemes to improve navigation focused on canals rather than rivers.

A large number of channelization schemes have been carried out for the purpose of flood alleviation or to improve agricultural drainage. From the period 1930 to 1980, 8500km of river were altered in such a way as to have a major lasting effect on channel morphology. This would typically include processes such as: embanking and embankment improvement, widening or deepening of the channel, realignment or bank protection. Figure 2.1 shows the extent of channelization schemes for this period.

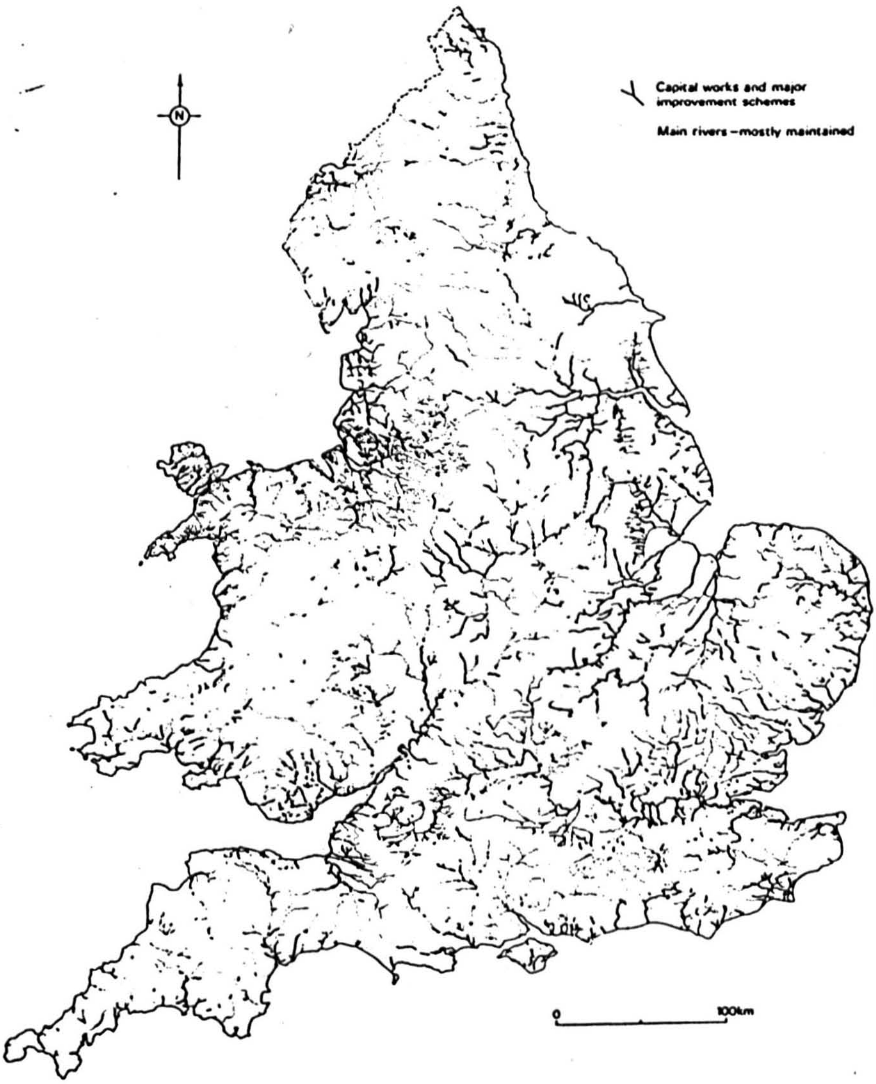


Figure 2.1. Rivers channelized in England and Wales 1930-1980. Taken from Brookes, 1988

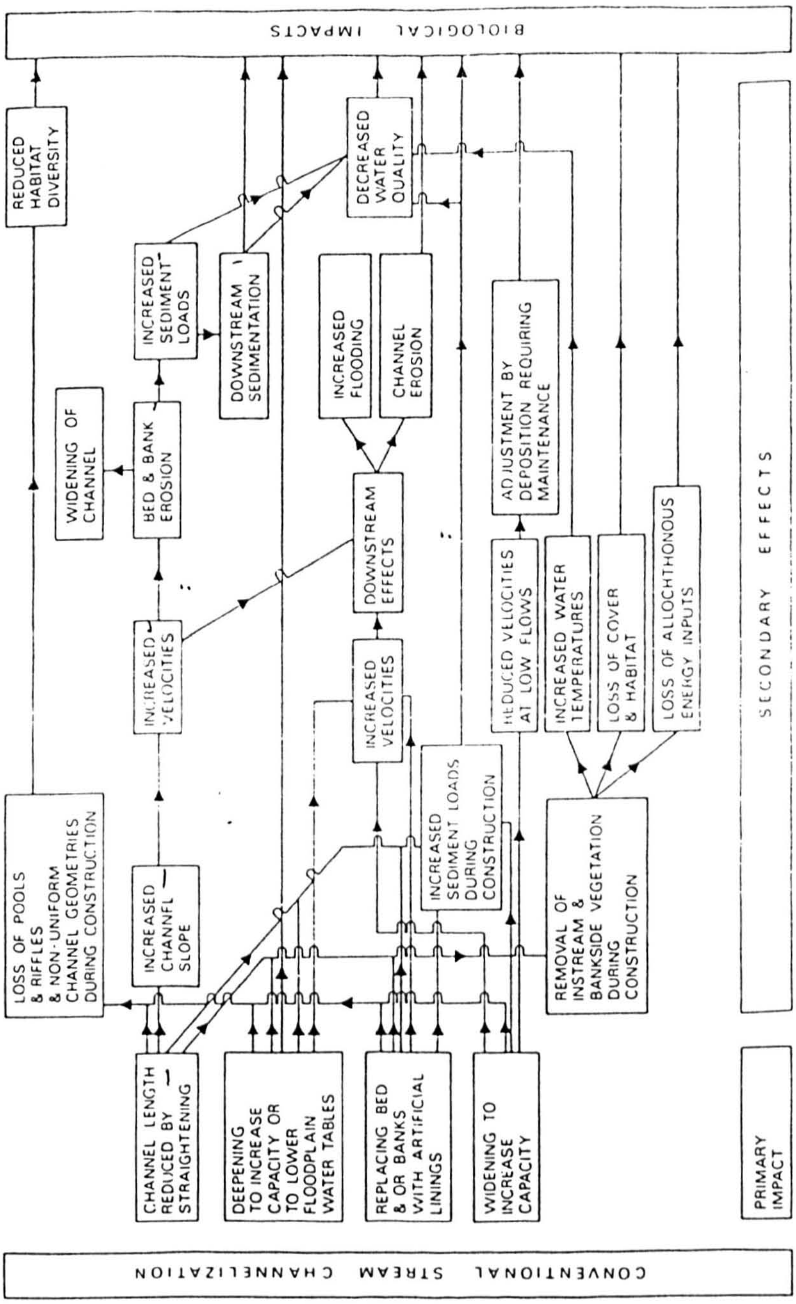


Figure 2.2. Impacts of Channelization. Taken from Brookes, 1988.

2.2.2 The Effects of Channelization

The main impacts of channelization are described by figure 2.2. It should be noted that rivers vary widely. As a result, the effects of a channelization scheme are hard to predict. An upland gravel bed river is unlikely to react in exactly the same way to a channelization scheme as a lowland silt bed one. In addition to this, a river can be in a stable or unstable condition. A river can be unstable due to the fact that it is reacting to some imposed change. An example of a possible cause of such instability could be a change in the land use surrounding the watercourse. As a result of these uncertainties, the statements which follow concerning the impacts of channelization are meant to be of a general nature. It is recommended that in the assessment of the site specific impacts of proposed or existing channelization, a morphological assessment of the river's condition is first employed.

Channelization can produce a wide range of biological impacts on benthic invertebrates, fish and aquatic vegetation. There are also significant effects on channel morphology. Both of these types of effects are discussed in more detail below. In general, channelization also produces a significant reduction in the aesthetic appearance of a watercourse. This can produce an associated reduction in amenity value as people do not wish to visit a river which they consider to be less attractive in appearance.

2.2.2.1 Physical Effects

A river is a dynamic system. Within that system there are continuous inputs of water and sediment (inflow hydrograph and sediment hydrograph). These inputs act on the channel boundary materials and vegetation and the result is a channel with a certain cross section, longitudinal profile and planform. Although the system is dynamic and constantly changing, it tends towards a condition of equilibrium. The equilibrium may not be a rigid condition but, for example, a particular sinuous planform shape may arise but the meanders may gradually migrate downstream. Channelization disrupts the equilibrium condition and there is, therefore, a good chance that this will promote adjustment in the channel to a new stable condition.

The type of channelization which often brings about the greatest physical readjustments is realignment. Here, shortening the path of a river increases the bed slope. This increases the sediment transport capacity of the river in the channelized reach. The river erodes its bed and banks to account for this. The degradation of the bed may be sufficient to undercut the banks and cause slips to occur. A nickpoint of erosion is produced and this can often migrate upstream. Downstream of the channelization, the sediment transport capacity is unaltered. As a result, the excess amount of material now being carried down from upstream cannot be transported, and it is deposited on the bed. This process is shown in figure 2.3. Protection of the riverbanks is often carried out in conjunction with straightening. Otherwise the river can erode its banks and return to a more sinuous path (Brookes, 1985).

Straightening a reach of river can cause flooding downstream. Flood hydrographs are attenuated to a greater extent when flowing around meanders than when traversing the shorter distance along a straight reach. Downstream flood defences may be unable to cope with the higher flood level.

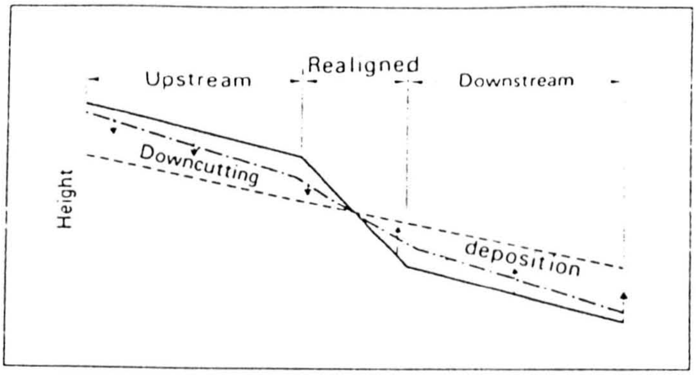


Figure 2.3. Degradation in Straightened Alluvial Channels. Taken from Brookes, 1988

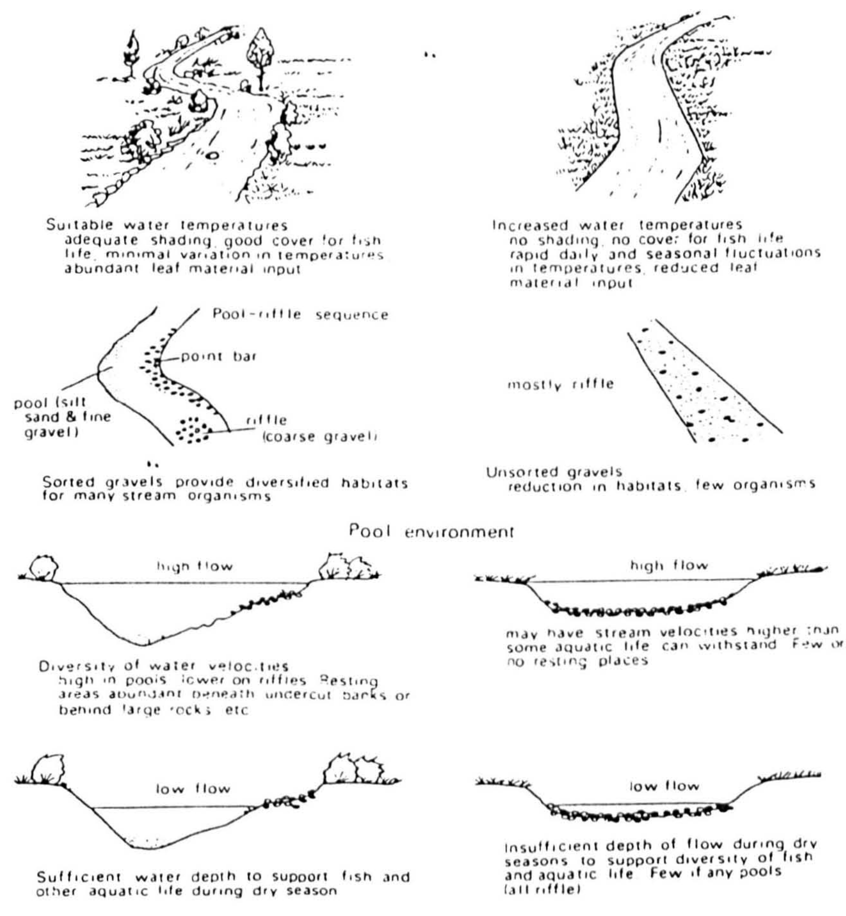


Figure 2.4. Comparison of the channel morphology and hydrology of a natural stream with a channelized watercourse. Taken from Brookes 1988.

Widening or deepening of a channel cross section increases the discharge capacity. This can put it out of equilibrium with the normal range of flows in the river. Widening reduces the velocity at low flow and reduces the sediment discharge. As a result, deposition occurs and this can create permanent morphological features. Over time, the deposited material can occupy a sufficient area of the channel such that it is returned to its old width.

Lined channels are less likely to adjust than channels that have been widened or deepened. On occasion, sediment can be deposited on the bed. This is most common, though, in channels that have been widened or deepened as well as being lined.

The construction of embankments means that larger discharges are contained than previously. These high discharges can generate larger velocities and, hence, degradation of the bed and banks can take place. Alternatively, sediment which would previously have been deposited on the floodplain during times of flood may begin to accumulate within the channel. Over time it may be necessary to continually increase the height of the flood defences to maintain the same level of flood defence.

The effects of clearing and snagging are the most difficult to assess. Trees retard bank erosion by stabilising the banks. However, fallen trees and log jams may cause localised erosion. Removing vegetation increases velocities and reduces the banks resistance to erosion. Clearing and snagging has been known to cause bank erosion and widening.

2.2.2.2 Biological Effects

Leaving aside the great disruption that construction process associated with channelization work can have, the lasting biological effects of the new design can be considerable. The main effects are shown in figure 2.4 and are discussed in more detail below.

2.2.2.2.1 Habitat Diversity

One of the main effects of channelization, in terms of the aquatic habitat, is the reduction in the complexity of habitat available. A natural, meandering river will have a varying channel cross section profile and, together with the sinuous planform, this produces a variety of habitat types. Typically, a channelized section of river is straight and has a uniform cross section. As a result, the flow and substrate tend to be very uniform throughout. In terms of the ecosystem this means that habitat diversity is reduced (habitat of a single type is provided). The geometry of a natural meandering river is shown in figure 2.5.

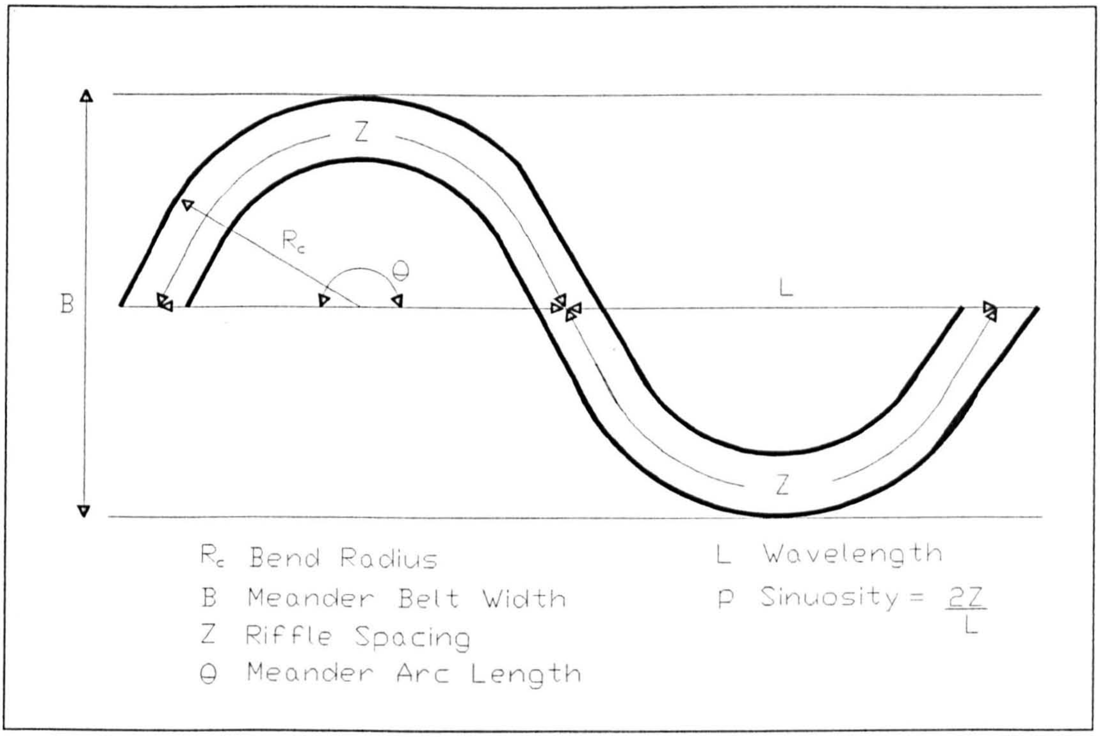


Figure 2.5. The geometry of a natural meandering river

It is believed that some species of fish require morphological diversity for feeding. Fish shelter in areas of low velocity, but move out into areas of high velocity to intercept passing food. The low velocity areas also provide shelter from the current during peak flows. Slack areas of flow are needed to provide suitable nursery areas for young fish.

A crucial part of habitat diversity is the pool riffle sequence, as highlighted n figure 2.4. A riffle is a section of faster flowing shallower water commonly found in a straight section of river between two bends. Riffles tend to have a gravely or stony substrate while pools have finer material on the bed. Pools tends to form on the outside of bends where the faster flowing water erodes the channel bed. As a result, the flow eventually becomes deeper and slower. The inside of a channel bend is an area of deposition where a point bar forms. The channel cross section is typically symmetric on the riffle and asymmetric in the pool (deeper at the outside of the bend). Pools and riffles provide a wide range of habitat types. Pools provide deep, slow moving areas of water often with bank cover. This is good habitat for sheltering from predators and resting. The fast flow on riffles has a greater oxygen content and is more suited for spawning. The importance of the pool riffle sequence was demonstrated by a study on the Olentangy River, Columbus, Ohio which showed that re-introduction of pools and riffles into a channelized river greatly increased the biomass of non-game fish (Edwards et al, 1984).

Observation of natural rivers reveal that pools commonly form at a spacing of 5-7 channel widths (Keller, 1978). The formation of pools and riffles is largely dependent on the channel slope, sediment concentration and bed material size. Keller found that pools and riffles were only evident in gravel bed rivers if the bed slope was less than 0.005. Thus, excessive shortening of a river during channelization will remove the possibility of the biologically important pools and riffles reforming in the altered channel. The pool riffle sequence was also severely affected by alterations to the channel width.

The most popular explanation for the maintenance of the pool riffle sequence in gravel bedded streams, and the associated sediment sorting which occurs, is the 'velocity-reversal hypothesis' (Carling, 1991). The theory is based on the assumption that at low flows the near bed velocity and bed shear stress are higher in a riffle than in a pool. As discharge increases the near bed velocity and bed shear stress increase more rapidly in the pool than in the riffle. Thus, at high flows the flow in the pool might be more competent in the pool than on the riffle. However tests by Keller (1971) failed to prove this.

Dolling (1968), using Duboy's equation, argued that the energy gradient (at first higher over the riffle) would equalise over undulating topography as depth increases. Thus, the shear stress would be larger in the pool at high flows as the depth would be greater. However this ignores the effect of local flow convergences and accelerations. Richards (1976, 1978) tested the theory but the results demonstrated that, although velocities became more closely matched at high flows, equalisation did not occur. However O'Conner et al (1986) demonstrated that, during a catastrophic flood in a bedrock canyon, the water surface slope in a riffle could become less than that over a pool.

The only studies which actually demonstrate a velocity and shear reversal in alluvial streams are those of Andrews (1979) and Lisle (1979) carried out on the East Fork River. These show that at one pool riffle pair a velocity reversal occurs with higher velocities in the pool than on the riffle at high flow. This is the opposite of the case at low flow. For discharge continuity, it must be the case that the cross sectional area at the riffle section is much higher at high flow than at the pool. This was accommodated by a 74% increase in cross section width in the above outlined experiment.

Thus the 'velocity reversal' theory relies on the fact that riffles are much wider than pools. Carling demonstrates on tests on the River Severn that no velocity reversal occurs, however an equalisation of the values of the average hydraulic variables did occur at high flow. In conclusion, Carling states that *'A detailed investigation of the three dimensional character of the flow is required to demonstrate whether the entrainment forces within pools can locally exceed those over neighbouring riffles'.*

2.2.2.2.2 Riparian and Wetland Habitat

The intermittent wetting of the floodplain can produce valuable wetland habitats adjacent to a watercourse. It is estimated that in the United States there is 50 million hectares of land that could potentially support a riparian ecosystem. Of this only 9 million hectares exists in a natural or semi-natural state. If flood defences are improved wetlands will gradually dry out. In this case, draining of wetlands can have serious impacts on the resident mammals, amphibians, insects and birds. Aquatic plants are often replaced by terrestrial species. Warblers and thrushes in particular thrive in the riparian habitat close to watercourses, and tend to be the worst affected by the removal of wetlands and the destruction of bank side flora and fauna.

2.2.2.2.3 Substrate

As has already been stated, channelized rivers tend to develop substrate of a single type due to the uniform nature of the flow. Substrate is a critical factor in habitat quality, especially for macroinvertebrates. A lot of these species live between the cracks in stones. Commonly, they require large stones and fast flowing water such as would be found on a natural riffle. Median bed material size has been found to be 1.47 time larger on the riffle than in pools (Hey and Thorne, 1986). Species such as mayflies, caddisflies and stoneflies require this type of habitat. The substrate in a channelized reach is often sand or a finer material, which is less productive for macroinvertebrates habitat. If any stony substrate does form, it can get choked with finer silt. Fish feed on macroinvertebrates and so are indirectly affected by alterations to the substrate.

Salmonids and some species of coarse fish require gravel on which to lay their eggs. The gravel must be free of silt so that water passes through it and brings oxygen to the eggs. This type of habitat occurs in riffles and sometimes at the tail end of pools in a natural river, but is unlikely to occur in a uniform channelized reach.

2.2.2.2.4 Cover

The shading effect of trees is important to control water temperature. Shading to deep pools, in particular, provides a habitat where fish can rest apart from the fastest part of the flow and away from direct sunlight. Removal of trees during channelization can be detrimental to habitat.

Unfortunately, it is not usually possible to incorporate the effect of cover into an assessment of aquatic habitat in a quantitative sense. Instead, some form of qualitative assessment of the available cover is usually all that can be achieved.

2.2.3 Previous Studies of Channelization Schemes

Brookes reports on a number of previous studies that have detailed the detrimental effects of channelization. Larimore and Smith found that continued dredging of streams removed pools and aquatic vegetation. This led to a drop in fish species from 90 to 74 over a 30 year period. Boussu found that removal of bush cover produced a 40.5% drop in fish population. Geghards found that water velocities rose following channelization, and discharges fluctuated more rapidly. Beland found that channelization of the Lower Colorado river changed a meandering stream with pools and riffles into a straight, dredged channel with turbid water. The loss of the pool riffle sequence was thought to be partly to blame for the decline in fish population found by Congdon on the Charlton River, Missouri. Elser found that the loss of 40% of the meanders on the Little Prickly Creek, Montana, together with the removal of the pool riffle sequence meant that 87% of the water was shallow and fast flowing post channelization, as opposed to 49% in unaltered reaches. The trout population declined by 78% in the altered reaches.

A very large number of studies reported drops in fish biomass following channelization, but did not link the changes to any specific factor. These included work by Trautman, Duvel et al, Norton et al, Beland, Daniels, Emerson, Barnard et al, Schummet al, Bird and Keller at al.. In some cases the effect has been so severe as to cause reductions in fish biomass of up to 100%. A study of the effect of 18 flood alleviation schemes in the UK found that the number of aquatic macrophytes were reduced following a mixture of dredging, widening and straightening schemes (Hey et al, 1994). Schemes involving two-stage channels or the construction of flood banks had no effect on the aquatic macrophytes.

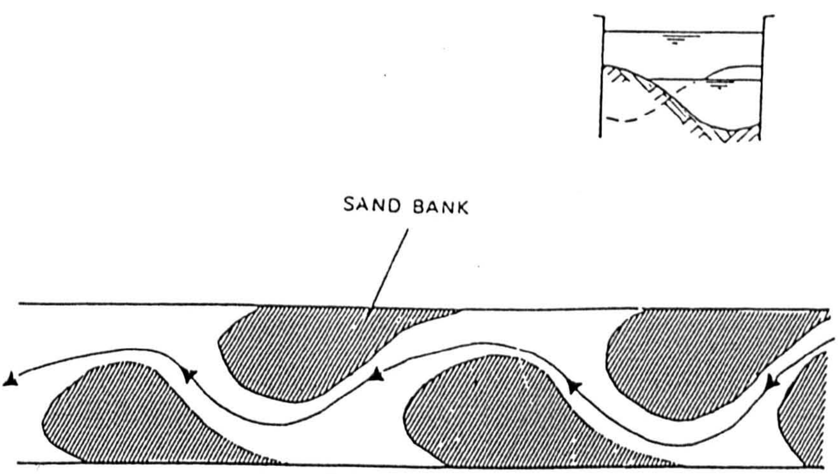


Figure 2.6. Propogation of sand banks in straight channels. Taken from Brookes, 1988

2.3. Natural Recovery

There are many recorded cases of channelized rivers turning to their original state through bed and bank erosion and deposition. Channels that have been both straightened and widened have been known to develop a sinuous thalweg, which forms a low flow channel within the main watercourse, figure 2.6. This process happens particularly on channels with high slopes. Brookes refers to the work of Tuckfield who observed adjustments in 53 New Forest streams in Hampshire following realignment and enlargement works carried out by the Forestry Commission. The channels on the steepest slopes showed the greatest amount of change. Brookes, reporting on the work of Nixon, showed that the widening of the River Tame near Birmingham resulted in the river reverting to its original capacity in less than 30 years.

Many channelization schemes involve some degree of straightening of the channel planform. Current thinking suggests that the size of meanders is a function of the discharge and channel width. Channels in coarser grained sediments tend to be wider and shallower and have larger meander wavelengths. This is because the banks are less cohesive and so become unstable and collapse at smaller heights than banks composed of silt and clay. Straightening disrupts the planform shape which will be at, or tending towards, a state of equilibrium. As a result, straightened channels tend to restore their former sinuosity. Shortening of the River Rhine between Basel and Strasbourg by 14% in the 1800's resulted in the river eroding a new set of meanders which were similar to the original planform. Brookes, referring to the work of Barnard and Melhorn, also found that the shortening of the Big Pine Creek in Indiana from 15.6 km to 10.9 km only had a short term effect as the river rapidly restored its previous sinuosity.

The extent to which a channel responds following straightening is significantly determined by the character of the bed and bank sediments. More erodible materials will permit large amounts of change to take place. Coarse segregated sediments or an armored layer of material will probably give rise to small amounts of erosion, and subsequent change, following straightening.

Brookes states that research by Brice highlighted three categories of factors important to bank stability following stream channelization. These were subdivided into site factors, alteration factors and post-alteration factors table 2.1. In addition, by considering 103 relocated channels in North America, Brice listed 13 factors contributing to stability of relocated channels and 15 factors responsible for instability. These are listed in order of importance in table 2.2.

Stream power is one of the most crucial variables in determining the stability of a redesigned watercourse following channelization (Brookes, 1987b). Stream power has been defined in the review of sediment transport modelling. It is highly dependent on the gradient and discharge. Channelized streams that have regained their former sinuosity following straightening have been found to have stream powers in excess of 100 W m^{-2}. Sites that have not restored their sinuosity but which exhibit bed and bank instability tend to have stream powers in excess of 35 W m^{-2}. Channels not adjusting by erosion tend to have stream powers below 35 W m^{-2}. In a further study, it was found that rivers with a mean bank full stream power of less than 15 W m^{-2} generally formed an environment where the processes of deposition were dominant, assuming that sufficient transported material was available from upstream (Brookes, 1990).

Recovery rates following channelization can vary widely. Brookes, referring to the work of De Vries, found that estimated recovery rates for five different rivers varied from 30 to 1000 years.

1. Site factors	Stream flow habit, drainage area, water discharge, channel width, bank height, sinuosity, stream type, valley relief, channel boundary material, incision of channel, vegetation cover along banks, prior channel stability, works of man
2. Alteration factors	Length of relocation, slope and cross-sections of relocated channels, aspects of channel alignment, measures for erosion control and environmental purposes
3. Post-alteration factors	Length of performance period, streamflow during performance period, post-construction maintenance and addition of countermeasures, growth of vegetation along the channel

Table 2.1. Factors important to the stability of relocated channels. Taken from Brookes, 1988

Stability		
1.	Growth of vegetation on banks	41 sites
2.	Bank revetment	33 sites
3.	Stability of prior channel	20 sites
4.	Straightness of channel	20 sites
5.	Low channel slope	16 sites
6.	Erosional resistance of bed or bank material	15 sites
7.	Minimal channel shortening	15 sites
8.	Bedrock control	13 sites
9.	Check dam or drop structure	11 sites
10.	Natural or artificial discharge regulation	10 sites
11.	Number of floods in first few years after construction	6 sites
12.	Preservation of original vegetation	3 sites
13.	Dual channel	3 sites
Instability		
1.	Bends in relocated channel	21 sites
2.	Floods of large recurrence interval	17 sites
3.	Erodibility of bed or bank materials	16 sites
4.	High channel side, susceptible to slumping	9 sites
5.	Instability of prior channel	8 sites
6.	Sharp decrease in channel length	8 sites
7.	Failure of revetment	7 sites
8.	Width-change factor too high or too low	6 sites
9.	Cleared field at bankline	5 sites
10.	Flood soon after construction	5 sites
11.	Lack of continuity in vegetal cover along banks	5 sites
12.	Turbulence at check dam or drop structure	4 sites
13.	Flow constriction at bridge	4 sites
14.	Non-linear junction with natural channel	3 sites
15.	Steep channel slope	2 sites

Table 2.2. Critical factors contributing to the stability and instability of relocated channels. Taken from Brookes, 1988

2.4. Rehabilitation and Restoration

The harmful physical and biological consequences of channelization have been outlined. A large number of techniques exist to counter these problems. Included in these methods are those that seek merely to mitigate against these effects in some way, or whose purpose is to promote the river to recover to something approaching its original pre-channelization state (rehabilitation), and also the complete removal of the channelization works and reinstatement of the original course of the watercourse (restoration). The adoption of these techniques marks a move away from the more traditional heavy handed engineering approach where a new design is imposed on the watercourse and towards methods which are more in tune with natural processes in the river. Here the intention is to create channel features that are enduring and in synergy with local flow processes. The purpose is to create meandering channels with pools and riffles to restore habitat features destroyed by channelization.

Large numbers of restoration projects have been carried out throughout the world. In order to highlight some of the varying approaches used, a small number of these schemes are briefly detailed below.

An 800m straightened stretch of the Stensbaek river in southern Jutland, Denmark (Brookes, 1987a) was restored after it was found that the channelization work was responsible for severe downcutting and bank collapse. The original course, substrate and dimensions of the river were restored. Trenches were excavated to reveal the dimensions and substrate of the old course, and historical maps were consulted to obtain the planform. Ecological and morphological studies following the work reveal that the design objectives are being met. A similar procedure was carried out on the Wandse river in Hamburb-Rahlstedt, Germany (Glitz, 1983). A kilometre of river was restored to its former course. Recollonation of the restored reach with fish was observed, and the public reaction to the scheme was favourable.

Keller (1978) used a novel approach to rehabilitation for a project on the Gum Branch stream near Charlotte, North Carolina. This stream had been channelized in 1974, and this had resulted in the accumulation of large amounts of sediment. Keller's scheme involved altering the cross section geometry of the river. The intention was to cause the stream to develop a number of point bars. No structures were used. A 130m stretch or river was reconstructed with steep banks on the outside of bends and shallow banks on the inside. The bank slopes were symmetrical where riffles were required. The scheme was successful in that large point bars built up through deposition of sediment on the inside of bends. These formed permanent features that were seen to survive a number of floods.

Before discussing the technical detail of various alleviation measures, consideration is given to the stability of these types of methods. In the review of channelization, it was shown that some of the physical effects that could result were considerable. The results of channelization can be large scale instability not only in the reach altered, but upstream and downstream as well. The results of this can be so severe that the river recovers to its pre-channelized state within a very short time scale. Of course, this means that the money spent on channelization is wasted. To ensure that the result of mitigation or restoration measures is not to cause other instability problems, consideration is now given to the subject of channel stability.

2.4.1 Constraints on River Restoration

In the discussion of the natural recovery of channels following restoration, the importance of the stream power and the composition of the bed and bank sediments were detailed. But why do rivers recover at all following channelization?

It has already been stated that within a channel there are continuous inputs of water and sediment (inflow hydrograph and sediment hydrograph) and that these inputs act on the channel boundary materials and vegetation, and the result is a channel with a certain cross section, longitudinal profile and planform. Rivers tend towards equilibrium so that if the input variables of discharge (Q), sediment load (Q_s), calibre of bed material (D), bank material, bank vegetation and valley slope (S_v) are held constant, then a stable condition will be obtained. When in equilibrium a river is said to be 'in regime'. Changing any of the inputs will cause the river to move towards a new regime channel geometry. If the river is in regime then the channel morphology will be uniquely defined by the values of the four input variables.

Channel geometry is defined by nine interrelated variables (also known as degrees of freedom): bankfull width (W), depth (d), maximum depth (d_m), height (Δ) and wavelength (λ) of bedforms, slope (S), velocity (V), sinuosity (p) and meander arc length (z) (Thorne et al, 1997). Channelization causes instability because it changes the values of one or more of these variables which, as has been stated, have been determined by the interaction of the four input variables. The result is that the channel attempts to restore equilibrium by readjusting one or more of its degrees of freedom.

It should be noted at this point that the input values of discharge and sediment load are in fact constantly changing. Research has shown that the bankfull discharge, together with the associated sediment load, produces the same channel geometry as the complete discharge and sediment hydrographs. Floods have a greater effect on the channel but are infrequent, whilst lower discharges have a lesser effect but are more common. The bankfull discharge is commonly taken to be the dominant, or channel forming, discharge which can be used in calculations of channel geometry. The return period of bankfull flow is often taken to be around 1.5 years.

Since there are nine degrees of freedom for channel change, to fully predict alterations in morphology would require nine process equations. These process equations are known as the rational equations. However, nine rational equations are not known so morphological change for a natural mobile bed river cannot be fully predicted. This is a crucial point and means that value judgements and past experience are required in predicting changes in hydraulic geometry. A common approach is to assume some of the degrees of freedom are fixed, or pre determined, as part of a design. As a result, fewer equations are needed. The process equations that can be employed include: discharge continuity, flow resistance and some type of sediment transport relationship. A fourth equation can be generated by using an extremal hypothesis. This is an assumption of some form regarding the governing processes in the river. Common extremal hypothesis are: 1) that the river adjusts its geometry to minimise the stream power. 2) The river adjusts its geometry, velocity and roughness to minimise the unit stream power required to transport a given sediment and water discharge. 3) The river adjusts to the point where the rate of energy dissipation is a minimum.

As an alternative to the use of rational equations, several researchers have attempted to derive empirically based equations to describe stable channel geometry. These are termed regime equations. The problem with all of these equations is that they are derived from a limited number of case scenarios, and so are limited in application to watercourses which are similar to those used to derive the equations. Among the most popular regime equations for mobile sand bed channels are the ones derived by Blench, and Simons and Albertson. For mobile gravel bed channels the most widely used equations are those published by: Hey and Thorne, Nixon, Kellerhals, Charlton et al, Bray, Hey and Andrews.

Two sets of equations, rational and regime, have been outlined which can be used in the design, or analysis of, stable channels. The limitations of the applications of both of these sets of equations have been outlined. As a result of the lack of a complete set of deterministic equations for predicting channel change, it is necessary to turn to other methods. The work carried out in chapter 6 highlights the use of a morphological study in conjunction with detailed modelling of the movement of sediments.

The morphological survey would focus on an assessment of many of the important factors, which have already been discussed as central to channel stability. This includes such elements as stream power and the composition of bed and bank sediments. In addition, it has been found that their exists a great difference in the effects of alterations between upland bedload transporting rivers, and lowland non bedload transporting rivers. Upland rivers have been found to be inherently less stable and are much less forgiving. Designs which are not in harmony with the natural condition are quickly destroyed (Hey, 1992). Lowland rivers, in general, offer the possibility of incorporating habitat features without too much fear of them being destroyed by river processes.

Perhaps the most important factor to consider when examining the likely effect of river engineering schemes is the existing condition of the river. This is one of the major factors to assess in the morphological assessment. Although a watercourse will tend towards a condition of equilibrium, it may not have attained it. This may be due to the fact that the watercourse is still reacting to a change in prevailing conditions. The river may be still be moving to a new equilibrium condition in response to, for example, long term climate change, the effects of increased urbanisation within the catchment or a landslide upstream (Simon, 1995). The only way to ascertain this is to carry out a morphological survey to examine the processes occurring within the river as a whole, and not just in the reach which is under consideration. Various means can be employed to determine whether the river is moving towards a new equilibrium or is in a stable, or quasi stable, condition. The most common approach is to consult old maps showing the rivers planform. The evolution of the planform, with time, can then be assessed. Other techniques include: continued site monitoring, dating of the sediment on the floodplain or using skill and judgement in conjunction with a detailed site survey.

The importance of carrying out a morphological survey to detect any existing instability, evaluate the dominant processes in the river, and to identify those reaches that would benefit most from restoration is discussed in more detail in chapter 6. Once this is completed, more detailed modelling of rehabilitation proposals can be carried out to determine their effect upon the erosion and deposition of sediments. Several models exist, or are under development, that seek to model river planform changes through the processes of erosion and deposition. However, none of the models that have so far been published can be considered to be generally valid and easy to use software packages (Mosselman, 1995). Many contain major assumptions, one of the most common being that the channel width remains constant throughout. In addition, these models usually calculate shear stresses (to determine the rate of erosion) from a simple formula, and not based on the predicted pattern of flow velocities in the channel. The two models, for the determination of the erosion and deposition of sediments, presented in chapter 6 do not seek to predict the longer term evolution of channel planform, but they can be used to gain a good appreciation of the localised areas in which erosion and deposition are likely to occur following rehabilitation.

2.4.1.1 The role of the engineer, the geomorphologist and the ecologist

Before beginning an account of possible rehabilitation techniques, and the methods employed on the rehabilitation of the River Idle in particular, it is worth examining the roles of the various professions in designing a scheme. It is argued here that input from civil engineers, geomorphologists and ecologists is required in order to arrive at an optimum rehabilitation design.

The ecologist is required to give expert advice, relevant to the design, on habitat for fish species, micro-organisms and vegetation. The civil engineer is responsible for determining the likely impact of alternative designs on flood defence capabilities. However, with the advent of habitat modelling, as detailed in chapter 5, the civil engineer is now becoming increasingly involved in assessing aquatic habitats. The current trend is towards using the output, in terms of predicted depths and velocities, from flow models to make habitat suitability predictions. This allows a choice to be made between alternative rehabilitation proposals on the basis of superior habitat creation, using only flow predictions from a hydraulic model. The possibilities presented by the use of habitat models is discussed in much more detail in chapter 5.

The concept of channel stability, and its importance in river rehabilitation, has already been discussed. The difficulty in determining what will constitute a stable design has also been mentioned. The lack of a comprehensive predictive model for channel change, and the potential drawbacks with the use of both the rational and regime equations, mean that the best approach is to draw upon the information that can be provided by both a geomorphologist and a civil engineer.

In assessing potential channel stability, the geomorphologist can make reference to both the regime and rational equations. However, the main tool is the use of a geomorhological survey to assess the river condition in detail. It is interesting to note that the geomorphologist considers the governing processes within the river as a whole, as well as at smaller scales, when considering stability. This gives a wider view of the current condition of the river. In contrast, the civil engineer is often involved in modelling in much more detail, and is often more focused on areas of particular interest. By combining the twin approaches it is possible to look at the specific detail of the effect of the flow structure in the area of concern, and within the context of the river as a whole entity. This matter is discussed in more detail in the introduction to chapter 6.

2.4.2 Rehabilitation Techniques

A large number of techniques can be employed in river rehabilitation. The major ones are outlined below, together with a description of the rehabilitation of the River Idle.

2.4.2.1 Weirs and dams

These structures improve habitat diversity by creating a backwater effect which is useful in generating an area of deep slow moving water upstream. Downstream the water is fast moving and typically erodes a deep pool of faster moving, well oxygenated water with a coarse grained bed. Material eroded from the pool can be deposited downstream to form a riffle. Brookes, commenting on the work of Gard, found that brook trout introduced into a Californian stream grew rapidly and reproduced over a four year period. No trout could survive in the river prior to the introduction of a number of weirs.

Weirs can also be used to create energy losses within the system. A variety of different designs are possible, figure 2.7. Natural materials, such as logs and rocks, can be used in the construction for the best aesthetics. Alternatively, sheet piles or concrete may be required.

2.4.2.2 Deflectors

Current deflectors provide a relatively cheap and effective means of mitigating against the harmful effects of channelization. They consist of structures built out into the channel to alter the flow pattern particularly at low flows. Deflectors can be used to deflect the current away from a bank to prevent erosion. However, they are generally drowned out at high flows. Deflectors provide both useful habitat features, and can help to promote the river to recover to a more natural state.

The deflector improves habitat diversity by constricting the channel width, and thereby creating a zone of faster flowing water. In the lee of the deflector a zone of slack water is created.

Eventually, the faster moving water adjacent to the tip of the deflector will scour a pool and this should help to develop a pool riffle sequence which will aid channel recovery. A typical effect of the installation of a deflector on bed topography is shown in figure 2.8. The slack zone of water behind the deflector will form a deposition area. Deflectors need to be installed in positions such that they reinforce the natural tendencies of the river. They need to be sited away from natural riffle sites, and be sufficiently low so that backwater effects do not drown out potential areas of riffle. Eventually, it is common for deflectors to become vegetated over and form a permanent part of the channel bank. Some typical deflector shapes are shown in figure 2.7. Brookes, in referring to an extensive study carried out by Hunt on the Lawrence Creek in Wisconsin, found that trout biomass increased considerably over a 5 year period following the installation of deflectors.

2.4.2.3 Vanes

Vanes are wholly submerged structures constructed on the channel bed for the purpose of promoting bed scour. They do this by enhancing secondary circulation in the flow.

Vanes are essentially wooden boards. They are constructed at an angle in the flow. This promotes the near bed flow to move in one direction, while the faster surface flow over spills the vane and moves in a different direction. The most common vane types are shown in figure 2.7 together with the effect they have on the development of pools and bars.

2.4.2.4 Pools and Riffles

Recreation of pools and riffles by dredging and dumping material is a good way to restore habitat diversity within a channelized reach, particularly in lowland rivers. The recreated pools and riffles need to be located in harmony with the existing meander pattern. Pools need to be at bends. The material that is dredged out can be used to create a point bar on the inside of the bend. Riffles need to be located between bends. The locations for the recreation of pools and riffles in a straight channelized reach are less easily determined and need careful consideration. Sometimes it is useful to construct the riffle at an angle across the river so that it directs the main force of the current towards the outside bank of the next bend. This will help to promote scour at the downstream bend and aid the growth of a pool.

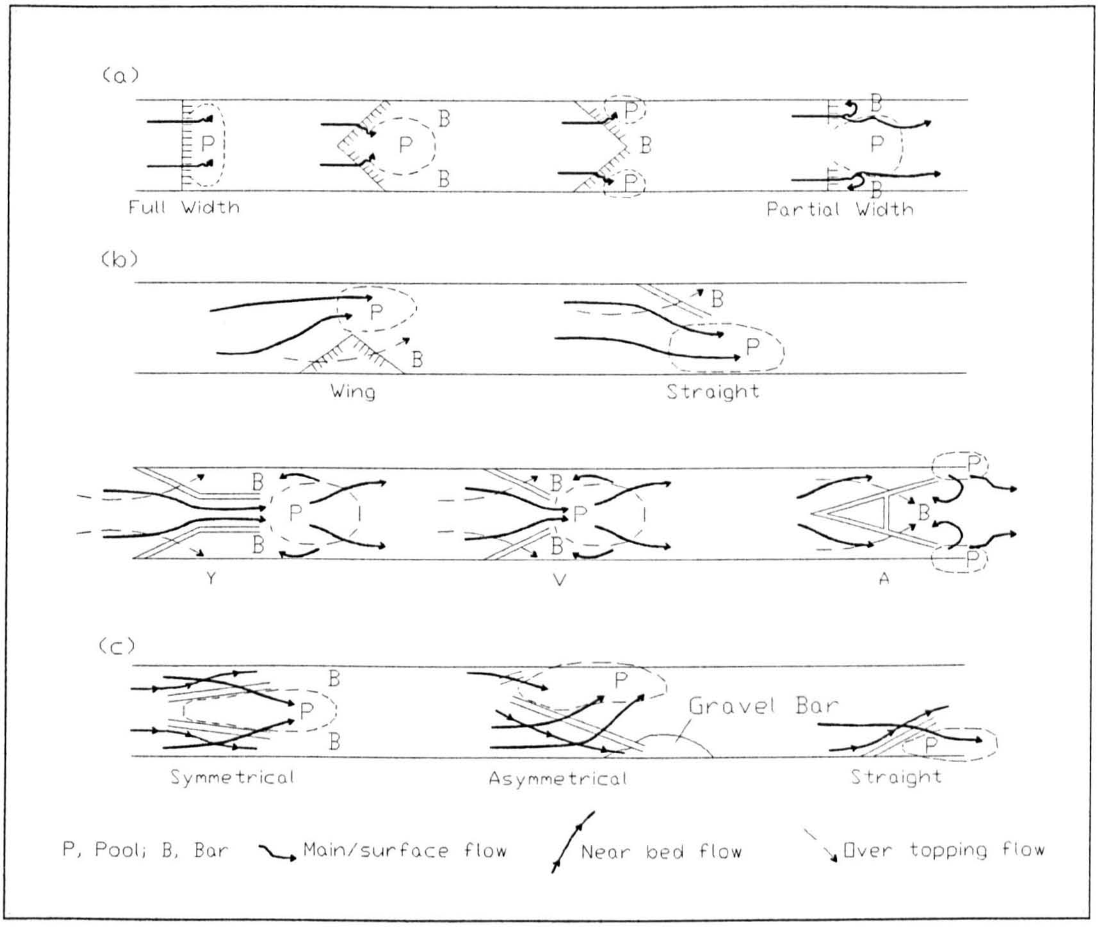


Figure 2.7. Designs for Weirs(a), Deflectors(b) and Vanes(c) with the associated flow patterns. Adapted from Thorne et al, 1997

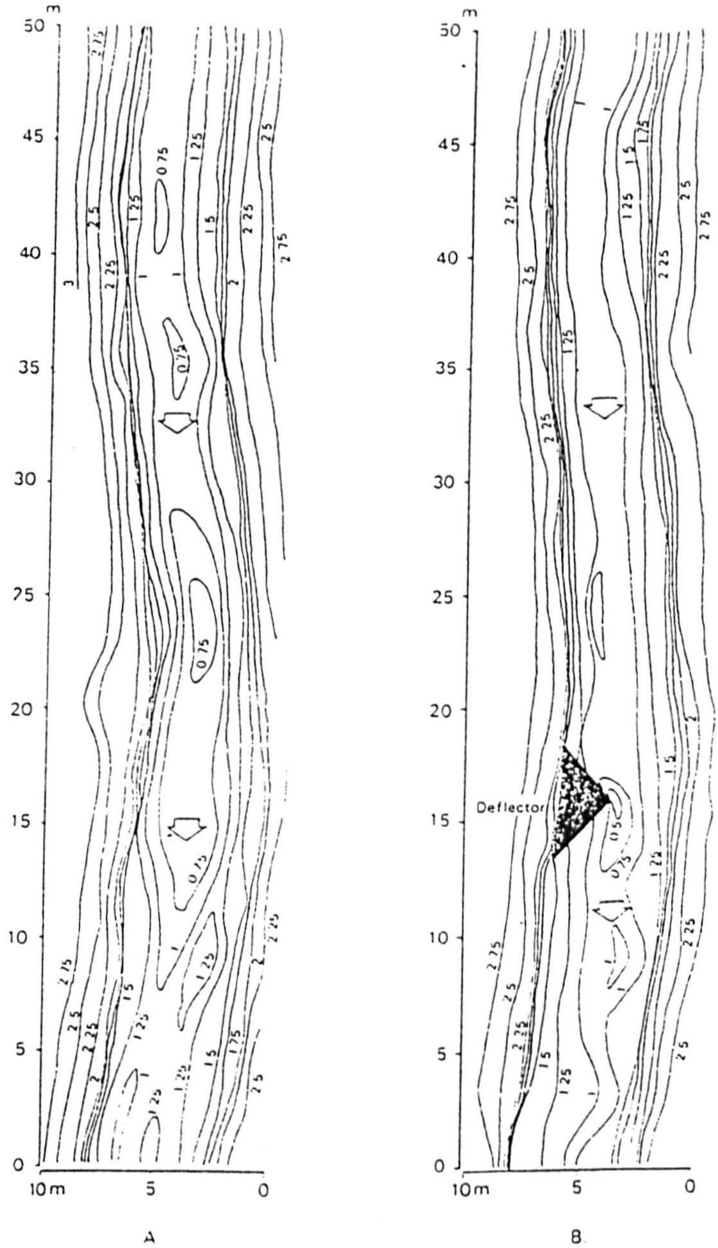


Figure 2.8. The impact of a deflector on the bed topography of a small stream in Denmark. (A) Before Installation and (B) formation of a pool one year after installation. Taken from Brookes, 1988

2.4.2.5 Substrate Reinstatement

Reinstatement of natural bed sediments can help to mitigate against the effects of channelization, and speed recovery. Well sorted gravels and cobbles or crushed rock placed on the bed can improve fish habitat considerably.

2.4.2.6 Restoration/Reconversion

An alternative to the methods outlined above, which seek to mitigate against the harmful effects of channelization and/or promote the river to recover to a more natural state, is to completely restore the channel to its prechannelized state. This approach may be necessary where the river has such low stream power that it would take an extreme amount of time before it fully recovered. The aim of river restoration is to recreate rivers for the benefit of wildlife and people. Although total restoration of a river is the usual aim of any scheme, financial constraints and the need to protect developments on the flood plain usually lead to some form of partial restoration with a reduced mandate.

The benefits of good restoration can be found in: nature conservation, improved fisheries, better water quality, improved flood defence and improved recreation facilities. Nature conservation comes through improved wildlife habitats. Restoration has a proven record of increasing the fish population (Biggs J., Williams P., not dated). Water quality is improved as the re-creation of natural flood plains helps to intercept pollutants, and by acting as a settlement area for flood-borne sediments. Restoring flood plains can aid in flood defence, by providing storage areas during floods. Studies have also shown a strong preference, in the general public, for natural landscapes.

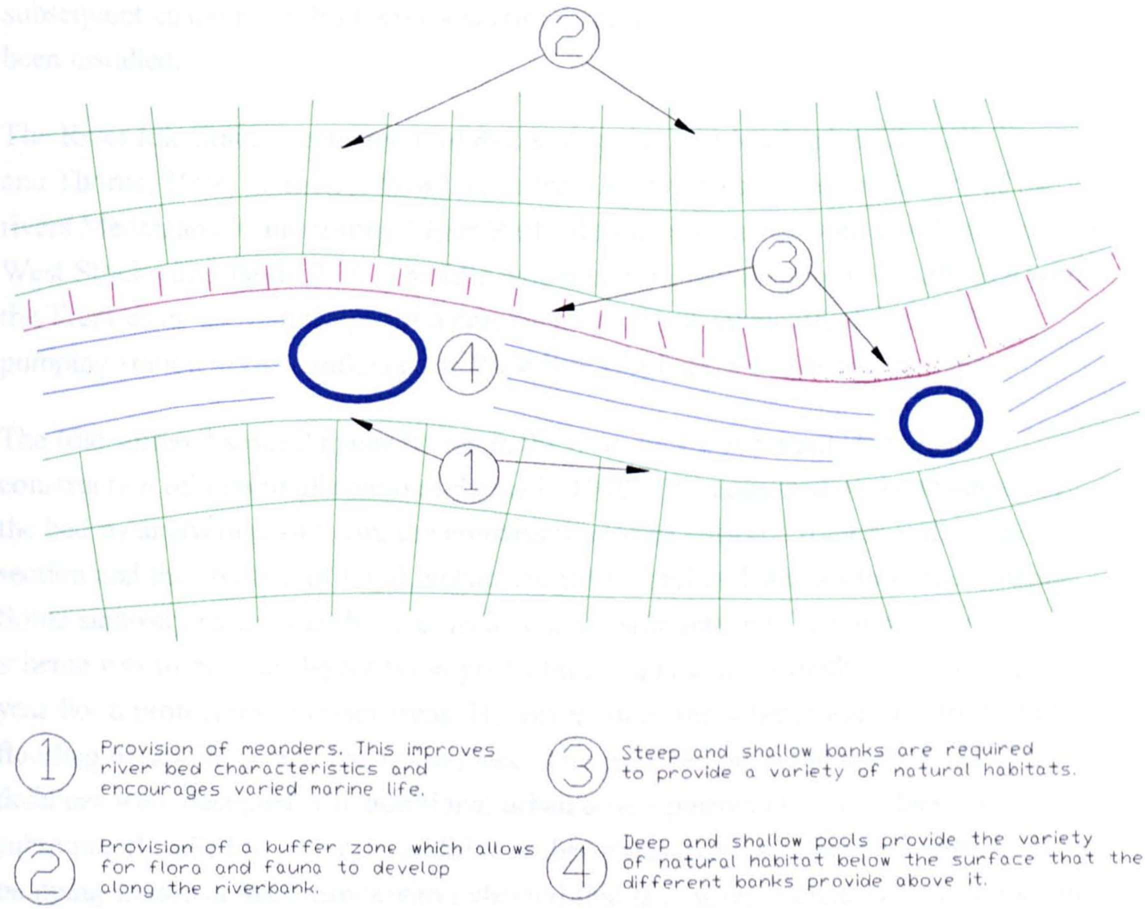


Figure 2.9: Stages in a Typical Restoration Scheme. Adapted from Biggs et al, not dated.

River restoration can include a wide variety of techniques, see figure 2.9. One of the most common, and most effective, is the provision of bends and meanders together with the recreation of the pool riffle sequence. This increases the river length and tends to slow the flow down. It is also improves habitat diversity.

The provision of buffer strips, adjacent to the river, provides a valuable wetland area for the growth of flora and fauna. The zone also provides a gap between the potentially damaging activities of man, and the river.

Steep and shallow banks are important to create a variety of habitats. Shallow banks are favored by many species of plant such as watercress, brooklime and water forget-me-not, while steep banks are used as nesting sites by birds such as kingfishers and sand martins.

2.4.3 Rehabilitation of the River Idle

The rehabilitation of the River Idle in North Nottinghamshire provides the focus for this thesis. It should be pointed out that all of the modelling work detailed in the subsequent chapters of this thesis was completed after the rehabilitation measures had been installed.

The River Idle drains a catchment of 842 km^2 in the East Midlands of the UK (Downs and Thorne, 1998). The river flows north from its its origin at the confluence of the rivers Meden and Maun south of East Retford to its confluence with the River Trent at West Stockwith, figure 2.10. The Idle would be tidal upstream of its confluence with the Trent as far as Mattersey, but a combination of embankments, sluices and a pumping station at the confluence at West Stockwith have eliminated this effect.

The Idle suffered several major floods in the past, which eventually led to the construction of a flood alleviation scheme in 1979. This consisted of the lowering of the bed by an average of 0.7m, the construction of an enlarged trapezoidal cross section and the erection of flood embankments at varying distances from the channel. Some sinuosity remains in the river as it was not straightened. The intention of the scheme was to provide 3-year flood protection to adjoining agricultural land, and 10-year flood protection to urban areas. However, since the scheme was constructed no flooding of adjoining land has taken place. This may be due to the fact that the flood defences were designed with additional urban developments in mind, which have subsequently failed to occur. In addition, subsequent analysis of the performance of the pumping station at West Stockwith indicated that the current defences over-service the river with respect to flood defence (Mott MacDonald Group, 1992, 1993). This gives some added flexibility for the rehabilitation design.

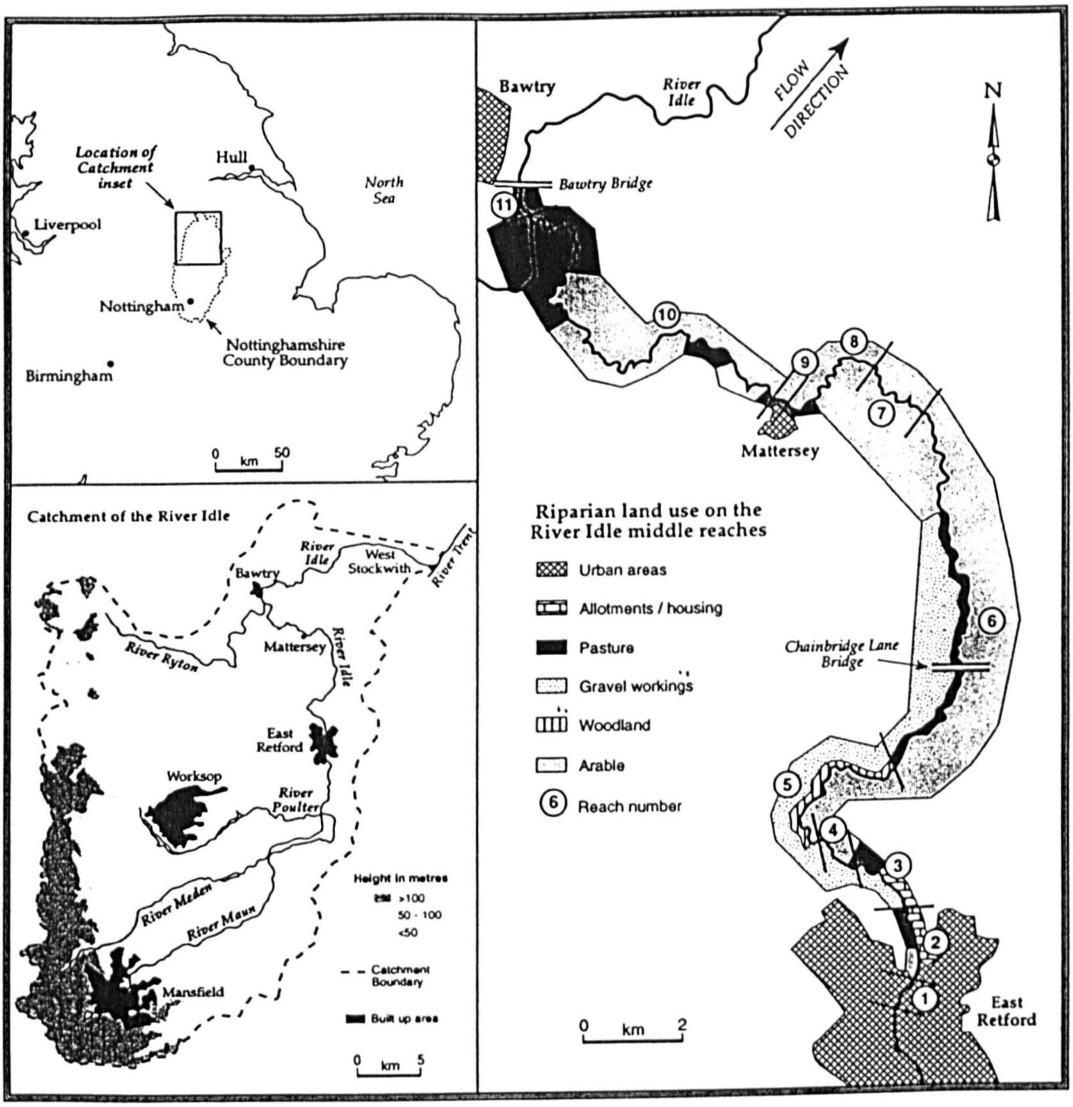


Figure 2.10. Location of the River Idle catchment within the UK: topography, settlement, and river network, and details of the project reach (Eat Retford to Bawtry). Project reach shows bordering floodplain land uses and division of the reach into sub-reaches on the basis of geomorphological conservation value.

The River Idle is a lowland, low energy channel with low stream power. The low gradient downstream of East Retford results in a bank full mean stream power of 0.6 to 2.2 Wm^{-2} (NRA, 1994) This lies well below the threshold for natural recovery of 35 Wm^{-2} discussed earlier in this chapter, and also below the threshold of 15 Wm^{-2} for a depositional environment (assuming there exists sufficient sediment from upstream). The process of deposition is manifest throughout the river in that it suffers from the accumulation of sand on the bed. Limited 'fingerprinting' of soils has confirmed that the sediment in the river derives from adjoining agricultural land. Eroded sediment arrives in the river either directly through aeolian transport or via drainage ditches. Surveys of the changes in channel capacity taken in 1979, 1982 and 1992 (table 2.3) show that the process of deposition is gradually returning the river to its pre-channelization state, but only at the rate of 1-2% loss of capacity annually (in the project reach at Bolham, Lound and Mattersey).

Section	Channel Capacity (m^2) (and % change)		
	Pre-scheme 1979	Scheme 1982	Post-scheme 1992
Bolham	77.0	90.8 (+18)	75.4 (-17)
Lound	53.5	63.0 (+18)	56.5 (-10)
Mattersey	28.0	40.0 (+43)	35.3 (-12)
Coneys Farm	44.6	52.8 (+18)	51.4 (-3)

Table 2.3. Channel capacity changes on the River Idle following channelization and subsequent sedimentation (taken from NRA, 1994)

As a result of the flood defence scheme, the Idle had only moderate amenity and conservation value but there is some angling interest. Electro-fishing surveys were carried out in 1989 and 1992, the results are shown in table 2.4. Species found to be present were: perch, bream, chub, dace, roach, eel, bleak, pike and gudgeon plus minor species such as stone loach. The results of the survey indicate a poor fishery both in total biomass and fish species present. The potential for improvement was noted in most sections. The presence of large amounts of sand on the river bed restricts feeding and spawning processes. In addition, the steep engineered banks restrict the amount of shallow water available at the channel margins for fry.

	Fish biomass – standing crop (g m^{-2})	
	1989	1992
Tiln Weir	4.0	-
Tiln Grange	-	9.01
Lound	4.0	-
Mattersey Priory	-	27.31
Bawtry	-	4.0

Table 2.4. Fish biomass in the River Idle obtained in two NRA electro-fishing surveys. Taken from Downs et al 1998

A geomorphological survey of the river was carried out by members of the School of Geography of The University of Nottingham in October 1994. Eleven reaches were identified which were considered to be 'in different ways geomorhologically homogenous'. The reaches are shown in figure 2.10. Bed material sampling revealed a gravel substrate overlain with large quantities of silty-sand. The conservation and rehabilitation potential of each reach was assessed using an approach developed for project appraisals (Downs and Brookes, 1994). The results are shown in table 2.5

Reach	Conservation Value	Rehabilitation Potential
1	1-channelized	Low
2	2-very poor	Medium
3	4-poor	Medium
4	2-very poor	Medium
5	5-moderate	Medium
6	2-very poor	Medium
7	4-poor	Medium
8	4-poor	Medium/low
9	2-very poor	Low
10	3-poor	Medium
11	2-very poor	Medium

Table 2.5. Summary of the results of the geomorphological evaluation of the River Idle (taken from Downs et al, 1998)

A survey of the recreational use of the river showed that it is used for angling, walking and canoeing. The results from a further investigation of the public perception of the river environment along the Idle, and the degree to which rehabilitation was desirable (Day, 1995), were in favour of reaches with higher geomorphological and ecological value. Thus, the rehabilitation of the river appears to be supported by the nearby population. The two typical reaches that were compared are shown in figures 2.11 and 2.12

Figure 2.11. View of River Idle in Reach 5 looking downstream. Obtained highest score in the river perception study of Day (1995). Highly valued due to sinuous planform, broken water surface, combination of light and shade on water surface and mature vegetation. Taken from Downs et al 1998. From black and white original.

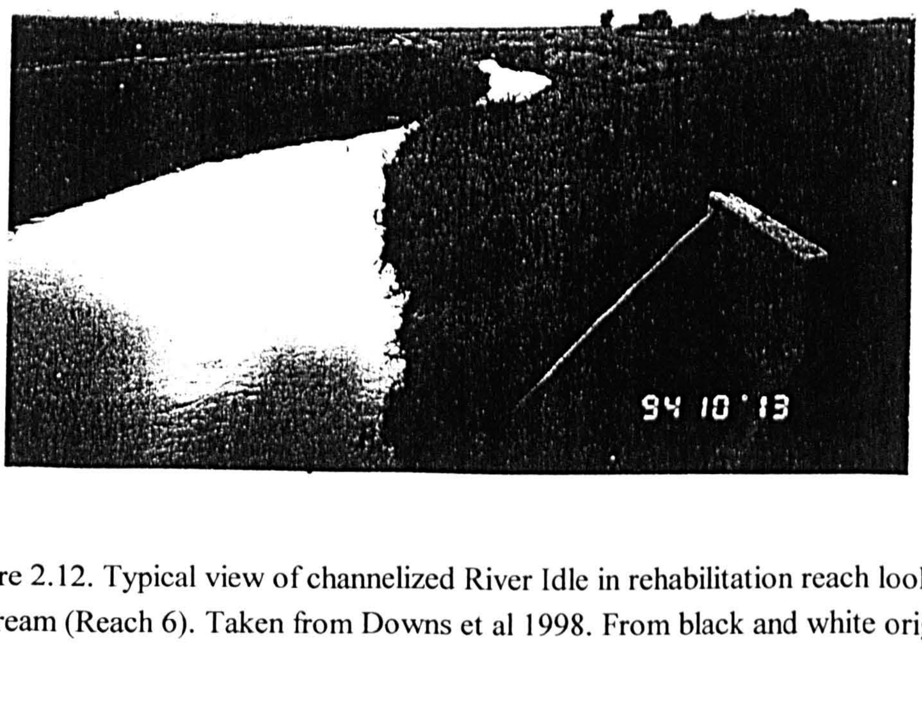


Figure 2.12. Typical view of channelized River Idle in rehabilitation reach looking upstream (Reach 6). Taken from Downs et al 1998. From black and white original.

The demands placed on the Idle for improved flood defence and land drainage resulted in the construction of a scheme that took no account of geomorphology, conservation or aesthetics. In addition, the unusually large extent of the channelization scheme has produced a low overall conservation value. A rehabilitation scheme was proposed for the river, with the requirement that it did not significantly increase the risk of flooding. The rehabilitation scheme was designed by Nottingham University Consultants Limited. Only a limited amount of channel recovery was to be permitted to occur, as too much could reduce the channel width and worsen flood conveyance. Some rehabilitation alternatives, such as the direct recreation of the pool/riffle sequence, could be eliminated immediately as they would do nothing to manage the large volumes of wind blown sediments arriving in the river.

Table 2.6. Four fold prioritisation of rehabilitation potential by sub-reach. Taken from Downs et al 1998.

Reach	Length (km)	Rehabilitation priority (1 = highest)	Comments on rehabilitation potential
1	0.4	4	Channelized and part of the urban fringe. Low priority as will require different approach to elsewhere in the project area.
2	0.75	3	Not embanked and is more constrained by land use. Possibilities exist for tree planting on the left bank. May be utilized for flood storage purposes at later dates.
3	1.2	3	As reach 2
4	0.7	4	Backs up behind Tiln weir. Leave at present and concentrate on reaches where flow characteristics can be harnessed to aid rehabilitation.
5	2.5	3	The geomorphologically most natural reach on the river and should be used for reference while measures are being introduced in other reaches. The existence of woodland and some cutbanks means that cliff-nesting bird species should be encouraged, and otter holts could be considered. The extent of tree-side shelter could be improved and bank top planting is recommended.
6	6.4	1	Very monotonous geomorphology and river corridor environment. Set-back embankments provide an opportunity for some corridor improvements. Prone to sedimentation. Existence of abutting wetland habitat and angling club interests suggest as the prime reach for rehabilitation measures.
7	1.3	1	As reach 6 but less obvious amenity benefits. Sinuous channel with embankments beyond meander limits provides good potential for corridor-based improvements.
8	1.9	2	Similar to reaches 6 and 7 but very low lying and prone to sedimentation and flooding. Rehabilitation measures for this reach will have to be especially careful not to increase flood levels. Existence of trees in river corridor reduces priority below reaches 6 and 7.
9	0.12	4	Maintain in present form for gauging purposes.
10	7.5	2	As reach 8 although less prone to flooding and sedimentation.
11	0.55	4	Low priority as NRA already experimenting with deflectors, and the confluence of River Ryton means reach is more typical of River Idle environments downstream of the project zone.

In table 2.6, the 11 geomorphologically defined reaches are categorised in more detail, in terms of their rehabilitation potential. Reach 6 was selected as the most suitable for rehabilitation due to the fact that it retains some natural sinuosity. There are also adjacent gravel workings along the left bank which reduce the economic consequences should flooding occur.

The river channelization scheme severed the natural link between the Idle and its floodplain. The lowering of the bed and construction of flood levees means that the natural sediment deposition on the floodplain, which should take place during flood events, will now occur much less frequently. It was not possible to rectify this situation without compromising flood defence requirements. Instead the rehabilitation was limited to the construction of in channel measures.

The final rehabilitation design consisted of the reprofiling of some bends, the planting of reed beds and trees and the construction of flow deflectors. By far the most significant changes to the river are associated with the installation of the flow deflectors. As a result, the work done in this thesis focuses on their effect on river levels and velocities, flow patterns, the movement of sediment and species habitat.

The deflectors use natural flow processes to achieve and promote habitat improvement by making local changes to the process of siltation.

The deflectors installed on the Idle consist of rock filled gabions. It was the intention of the design that the deflectors would become overgrown with vegetation and form permanent features in the riverbank. A number of different deflector designs were installed, and each one was constructed at a location where it would enhance the natural tendencies of the river by taking advantage of the remaining sinuosity in the channel. The purpose of installing the deflectors was to encourage the deposition of sediment to form a sand bar in the lee of the deflector (at the inside of the bend). At the same time a deeper, narrower pool would be eroded adjacent to the tip of the deflector (at the outside of the bend). Eventually, this should lead to the creation of a more meandering thalweg within a narrower channel. In addition, the deflectors should increase the stream power and this will assist in transporting the sediment through the river. It was believed that the velocities in the newly created pool would be sufficient to erode the sand from the bed and expose the underlying gravel. This would contribute to habitat diversity. The rehabilitation proposals are shown in figure 2.13. A number of the deflectors were subsequently constructed at the incorrect location so the plan provided does not accurately represent what was constructed on site.

The requirements of this rehabilitation can be said to be '*to provide a gradual and sustainable improvement to a severely degraded system within a multi-functional management framework, designed to avoid compromising existing or proposed* *functions'* (Downs et al, 1995). The intention was to create some variability within the channel cross section without promoting excessive bank erosion, which could eventually compromise flood defence requirements. The installation of the deflectors should promote the recovery of the natural pool riffle sequence leading to improved aquatic habitat.

No standard practices or design standards exist for determining the detail of deflectors. As a result, a number of different sizes and shapes of deflector were installed to test their relative effectiveness. Some photographs of the deflectors installed in the River Idle are shown in Appendix A.

Although 'success criteria' for the River Idle rehabilitation are not clearly defined, they include the implicit desire to increase the fisheries value and reduce the morphological monotony of the channel. It is crucial to monitor the effect of the installation of the deflectors, as part of a post project appraisal, to see if these criteria have been met. A thesis is ongoing, in parallel with this one, that will report on the changes that have taken place in the Idle since the deflectors were installed.

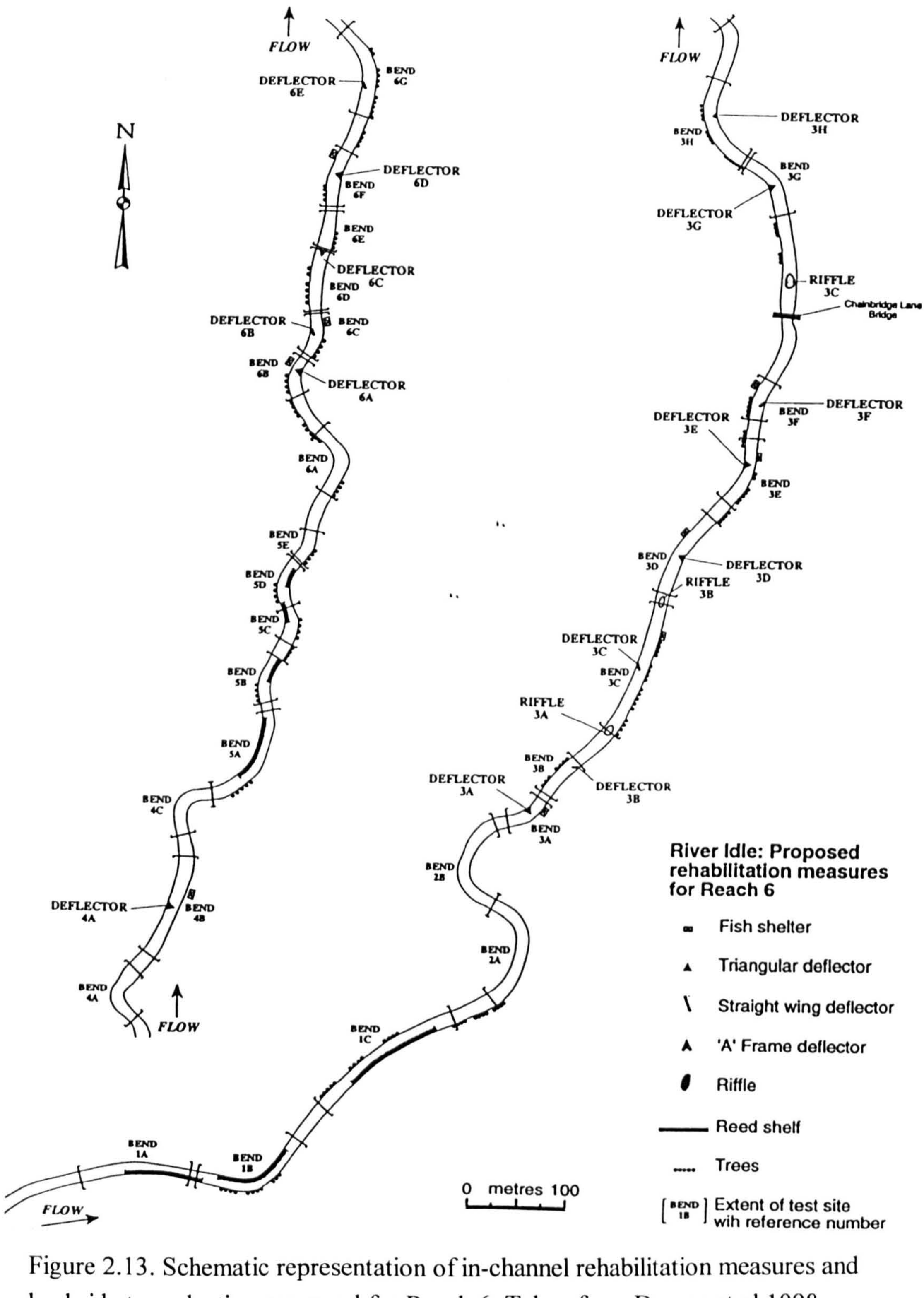


Figure 2.13. Schematic representation of in-channel rehabilitation measures and bankside tree planting proposed for Reach 6. Taken from Downs et al 1998.

2.4.5 Previous Restoration and Rehabilitation Schemes

Large numbers of restoration projects have been carried out throughout the world. In order to highlight some of the varying approaches used, a small number of these schemes are briefly detailed below.

An 800m straightened stretch of the Stensbaek river in southern Jutland, Denmark (Brookes, 1987a) was restored after it was found that the channelization work was responsible for severe downcutting and bank collapse. The original course, substrate and dimensions of the river were restored. Trenches were excavated to reveal the dimensions and substrate of the old course, and historical maps were consulted to obtain the planform. Ecological and morphological studies following the work reveal that the design objectives are being met. A similar procedure was carried out on the Wandse river in Hamburb-Rahlstedt, Germany (Glitz, 1983). A kilometre of river was restored to its former course. Recollonation of the restored reach with fish was observed, and the public reaction to the scheme was favourable.

Keller used a novel approach to rehabilitation for a project on the Gum Branch stream near Charlotte, North Carolina. This stream had been channelized in 1974, and this had resulted in the accumulation of large amounts of sediment. Keller's scheme involved altering the cross section geometry of the river. The intention was to cause the stream to develop a number of point bars. No structures were used. A 130m stretch or river was reconstructed with steep banks on the outside of bends and shallow banks on the inside. The bank slopes were symmetrical where riffles were required. The scheme was successful in that large point bars built up through deposition of sediment on the inside of bends. These formed permanent features that were seen to survive a number of floods

Chapter 3

Review of Flow Modelling

3.1. Introduction

The first question to address in a chapter on flow modelling is what is a model? A model can be defined as a conceptual representation of a structure or process, containing all the essential features of the object that it represents. In the case of river models, this will need to include flood plains, weirs, gates etc. A model can be anywhere from extremely simple, to highly complex. They can be mathematical, physical or conceptual. The first models were used simply to automate certain procedures. Nowadays, models are more complex, and can be used throughout the appraisal and design of any project. In effect, they have become fully interactive decision support tools.

It is crucial, in the construction and use of any model, that the modeller is aware of the limitations, hypotheses and structure of the modelling system being used. If the modeller is not aware of this, it typically leads to a 'black box' approach (Cunge and Verwey 1980), where the user accepts the output from the program without question. It should also be noted that the successful application of a numerical model, to any problem, is limited by the skill with which the modeller schematises the system. This manifests itself in the choice of cross sections, how storage is incorporated in the model, selection of boundary conditions, calibration of bed roughness etc. Poor selection and representation of data in the model makes the production of good results virtually impossible.

One other factor to consider in numerical modelling, is the balance between cost and accuracy of the model that is used. There are a wide variety of flow models available, each giving a different quality of results. It is necessary to balance the cost of the model employed (both in initial outlay, and in terms of the man hours taken to become familiar with the model and input all the data) with the use it will be put to. There is little point employing the most expensive model to simulate the effect of a minute alteration to a watercourse. Similarly, the results from the cheapest available model may not provide a sufficient basis on which to design a multi-million pound improvement scheme.

3.1.1 Application of Flow Models

In order to examine the possible effects of any proposed alteration to a watercourse on flooding extent and frequency it is necessary to have an accurate flow model. If the incoming discharge under consideration is constant then a steady state model will suffice. In the case where the engineer wishes to model the propagation of a flood wave down a watercourse, an unsteady model is required. In the unsteady case, the engineer will require information on how fast the wave will propagate downstream, and at what depth. This information may be required in order to design efficient flood defence systems and operate an effective flood warning system, for example.

Computational computer based flow models can be used to reproduce past flood events, and to simulate the effect that they had on catchments. For this purpose, a bank of data on past flood events, with measured rainfalls, plus river discharges and levels, is crucial to the calibration of any model. Once the computer model is able to accurately reproduce the effect that observed storms had on catchments, it can then be used in the assessment of the effect of alterations to the river.

The National Rivers Authority (NRA) was established in 1989, with responsibility to protect and improve the water environment. In 1996, this organisation became the Environment Agency (EA). The key areas of responsibility of the EA are: water quality, water resources, flood defence , recreation, navigation, conservation, and fisheries. Of these, flood defence is the key function, and this accounts for over half of all expenditure. In 1992-93 the National Rivers Authority spent £209M on flood defence, of which £112M went on capital schemes , and £88M on maintenance works (Pickles, 1993). Flood defence can be split into the actual provision, and maintenance, of defences, and the operation of a flood forecasting and warning system.

The major way that the EA, or its consulting engineers, uses computer flow models is in the design, and management, of new flood defences, or in the assessment of the effect of any alteration to the watercourse on flooding potential. The use of models makes comparing the likely performance of design alternatives much easier. In the past, local judgement has been used to decide relative scheme priorities. Using 'design' storms it is possible to easily compare the benefits of each scheme.

The actual models used by the EA, are many and varied. For the analysis of entire catchments, for assessing overall flood defences, a very generalised model is usually used, based on the Flood Studies Report (FSR) which is discussed later. This can be used to identify key points in a watercourse network. For the forecasting of floods for an entire catchment, a hydrological model may be used.

Sub-catchments may be modelled using simple FSR methods in rural areas. In addition, simple flood routing techniques are often employed using recorded hydrographs. Some form of computational backwater program can be used to good affect for some sub-catchments, where backwater effects are known to be significant.

For the actual design of schemes, or analysis of watercourse alterations, 1D computational modelling packages are the most widely used. Two and three dimensional packages may be used in situations where the detail of the flow structure is important.

3.1.2 Scope of This Chapter

This chapter describes the underlying theory behind one, two and three dimensional modelling. The various packages that were used in this thesis are also introduced and discussed.

The chapter begins with a description of the means by which a one dimensional hydraulic model can route a hydrograph down the river using the St. Venant equations. A number of techniques can be employed to solve these equations, such as finite difference and method of characteristics approaches. These techniques are outlined. Two one-dimensional hydraulic modelling packages, ISIS and HEC-RAS, are discussed. These form the basis of the one dimensional work carried out in this thesis.

Recently, a number of two and three dimensional modelling packages have emerged to compete with the one-dimensional hydraulic analysis software. Among these are SSIIM and CFX, both of which are outlined later in this chapter. These packages offer the ability to predict velocities and depths with greater spatial detail and accuracy. However, many two and three dimensional packages are not specifically targeted at the river modelling community. Partly as a result of this, they often require the user to be more experienced in computational modelling.

One dimensional models are widely used in general practice by the river modelling community. The use of two and three dimensional models is generally restricted to specialist applications, where it is necessary to model the flow field in much greater detail. The work carried out in this thesis focuses on the results that can be obtained using the different approaches, and their application not only to flow problems but also to the analysis of aquatic habitats and the movement of sediment.

3.2. One Dimensional Modelling

Some relatively basic techniques, often referred to as simple flood routing methods, are available to solve open channel flood problems. These involve some form of statistical analysis of the passage of historical floods, in combination with empirical relations, to solve flow problems. An analysis of these types of method is beyond the scope of this thesis. A number of texts deal with these types of approach such as the textbook of Chadwick and Morfett, 1986. The main alternative to these simple flood routing methods is to employ a fully hydrodynamic, one dimensional model. This involves solving two equations simultaneously. The first equation represents conservation of mass, whilst the second is either a momentum or energy balance equation. However, since it is possible to fully describe one dimensional unsteady flows with any two dependent variables chosen from discharge, depth and velocity, only two separate equations are needed. The two that are normally chosen are for the conservation of mass and a momentum balance equation, and together these are known as the St. Venant equations, equations 3-1 and 3-2. The version of the equations given below is based on a constant channel width:

(3-1) Conservation of mass: $$\frac{\partial Q}{\partial x} + B\frac{\partial y}{\partial t} = 0$$

(3-2) Momentum balance: $$S_f = S_o - \frac{\partial y}{\partial x} - \frac{u}{g}\frac{\partial u}{\partial x} - \frac{1}{g}\frac{\partial u}{\partial t}$$

Q = discharge, x = longitudinal distance along channel, B = breadth of channel, y = depth, t = time, S_f = friction slope, S_o = bed slope, u = velocity, g = acceleration due to gravity

The St. Venant equations include only the most significant parameters in hydraulic analysis of unsteady flows. Many subsidiary effects are not included for the sake of retaining a degree of simplicity. There are several assumptions in the equations (Cunge, 1980):

(a) One dimensional flow exists, with uniform velocity over the cross section and a horizontal water surface across the section.

(b) Vertical accelerations are negligible. This allows the assumption of a hydrostatic pressure distribution and relatively small streamline curvature.

(c) Resistance terms can be represented in the same way as they are in steady state problems.

(d) Small bed slopes.

True one dimensional flow does not exist. However, if the above limitations are met, then analysis using the St. Venant equations will, in many cases, be sufficient to produce a meaningful representation of the flow in the channel under consideration.

Most real life watercourses are not easily accommodated by the original form of the St. Venant equations quoted, with their associated rigid limitations for use. As a result, much work has been done to introduce factors into the equations to make them more generally applicable to natural channels. One of the most common modifications occurs when considering an area of flood plain which acts as a storage area only, in times of flood, but does not contribute to the discharge in the main part of the channel. In this situation, it is common to replace the channel width term B, by a 'storage width' term B_{st}, based on the width available for storage (including the flood plain), where B_{st} is a function of the depth. This allows the user to input an irregular cross section shape. Another possible modification is to include a term (q) for a continuous lateral inflow (or outflow), along a section, into the mass conservation equation. This additional flow may come from a tributary or due to natural processes such as evaporation, rainfall and infiltration, which affect the main watercourse in the area under consideration.

The resistance terms, in the St. Venant equations, are based on the empirical Chezy or Manning equation, more commonly associated with analysis of steady flows. The equations require one value of roughness to be taken for the entire cross section, and this is too much of a deviation from reality in many cases, particularly for compound channels (those with in bank and out of bank flows). In these channels, the roughness of the flood plain is likely to be considerably more than the main channel. To overcome this, many packages split the cross section into vertical slices, each of which has its own value of roughness. It continues to be necessary to maintain a horizontal free water surface across the section, and an identical friction slope (S_f) for each slice.

The St. Venant equations are not directly soluble in the above form, but approximate solutions can be obtained by the use of computers in conjunction with numerical schemes. Numerical solutions are obtained by discretizing the two laws in order to approximate values of the dependant variables at a finite number of points. Three distinct numerical methods of solution exist: finite element methods(FEM), finite difference methods (FDM), and the method of characteristics.

3.2.1 Finite Element and Finite Difference Methods

FEM methods have only recently begun to be applied to open channel flow problems, and their value is not yet discernible. They are discussed briefly in the review of three dimensional modelling techniques later in this chapter.

Finite difference methods are used in the majority of industrial models, including the ISIS package. Here, a region of solution in space and time (x,t) is defined which has a finite number of solution points. Within this region the derivatives in the St. Venant equations are replaced by discrete functions based on the numerical scheme used. An example is shown in figure 3.1. This is the particular form of discretisation used in the ISIS open channel program which is discussed later in this chapter.

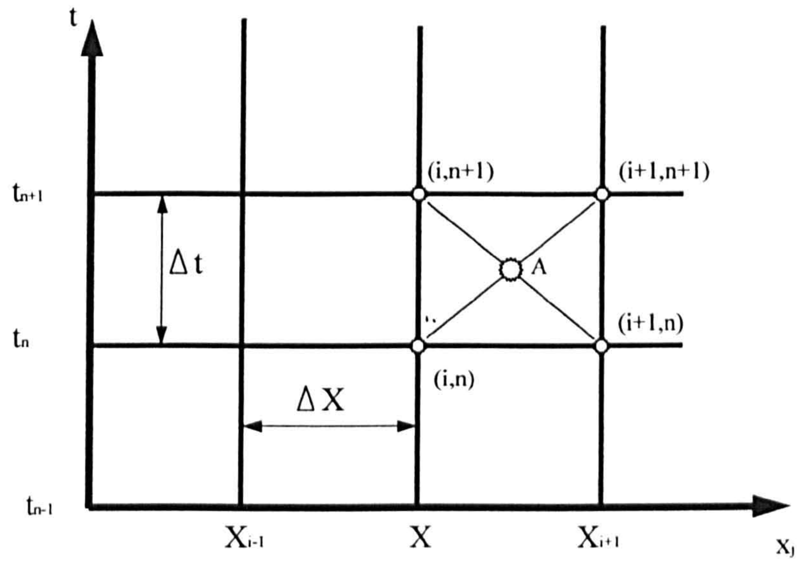


Figure 3.1 The Preissman 4-Point Implicit Box Scheme

The space derivative for the point A in this discretisation is given by equation 3-3:

$$\left(\frac{df}{dx}\right)_A = \frac{1}{2\Delta x}\left(f_{i+1}^{n+1} - f_i^{n+1} + f_{i+1}^{n} - f_i^{n}\right) \quad (3\text{-}3)$$

The time derivative for the point A in this discretisation is given by equation 3-4:

$$\left(\frac{df}{dt}\right)_A = \frac{1}{2\Delta t}\left(f_{i+1}^{n+1} - f_{i+1}^{n} + f_i^{n+1} - f_i^{n}\right) \quad (3\text{-}4)$$

The Preissman scheme is an example of an implicit scheme, which means that the derivatives contain values at the next time level (n+1), as well as the current time level (n). As a result, the solution process reveals a full set of simultaneous equations for values at n+1, which are required to be solved simultaneously. This is the opposite of an explicit scheme, where the values at any point on the next time level, are related only to values at the current time level and, hence, each point at the next time level can be treated separately from the remaining values at the following time level.

The Preissman scheme is solved in an iterative manner. Values at two time levels are continuously iterated in a loop until convergence is obtained.

Implicit schemes are not subject to the Courant restraint (see later), on the size of the time step employed, which affects explicit schemes. This is a major advantage as many explicit schemes need to employ extremely small time steps to remain stable. These small time steps can lead to very large computational times. As a result, implicit schemes are becoming increasingly popular. However, there are some disadvantages with this approach. The computations carried out are more complex than for an explicit scheme. In addition, time steps with implicit schemes have to remain sufficiently small to ensure that the solution is accurate.

A number of discretisation schemes can be applied when using a finite difference approach. Common discretisation schemes include upwinding and central differencing. Basically this determines what the finite difference stencil looks like. The stencil determines the way in which values of the variables at each location in time and space, within the solution, are located to values of the variables at adjoining points. The Preissman scheme is an example of one type of stencil. Discretisation is discussed in a little more detail in the later section on three dimensional modelling.

3.2.2 Method of Characteristic Lines

This method of solving the shallow water equations can produce explicit solutions to the equations, in some simple cases. It is also amenable to numerical methods in more complex cases, where explicit solutions are not readily available.

3.2.2.1 Flow Regime and Boundary Conditions

The type of boundary conditions that have to be specified in order to solve a flow problem are largely dependant upon the flow regime that is to be solved. In subcritical flow, information is propagated in both the upstream and downstream directions. As a result it is necessary to specify some form of boundary condition at either end of the flow domain. Commonly a discharge versus time relation is specified at the upstream boundary, and either a head versus time or head versus discharge relationship is specified at the downstream boundary.

In supercritical flow all information is propagated in the downstream direction. As a result, it is not necessary to specify any information at the downstream boundary, as it can have no effect on flow conditions upstream. Instead, two independent boundary conditions are needed at the upstream boundary; one for each of the dependant variables.

3.3. Commercially Available Software Packages

A number of packages are available for the solution of one dimensional flow problems. Only ISIS and HEC-RAS are discussed here, as they form the basis for the work carried out as part of this thesis.

Since one dimensional models are widely used by river modellers in industry it is perhaps tempting to think that the results that they produce are always wholly reliable. The report into the floods that took place in Easter, 1998 pointed out that this is not always the case. This is highlighted by the following quote:

'*The one area of particular concern is the application of computational hydraulic modelling in the design process to situations where the complexities of the flow behaviour are beyond the limitations of the theoretical and empirical concepts on which the models are based*' (Independent review team, 1998).

The above report points out the fact that predictions from one dimensional models can be inaccurate where the results have not been properly validated against recorded flood data. In addition, the report highlights the fact that the performance of structures in the channel, are often not accurately represented in the model. The need to specify roughness and discharge coefficients in the model can also be a source of error.

A need still exists to develop one dimensional modelling packages so that the reliability of predictions can be increased.

3.3.1 ISIS

ISIS is a commercial package jointly produced by HR Wallingford Ltd. and Sir William Halcrow & Partners Ltd. The package can simulate both steady and unsteady flows. It provides a choice of methods to model the propagation of the flood wave downstream, including several variations of simple flood routing methods. Most commonly, though, a fully dynamic approach is used based on numerical solution of the St Venant equations.

The ISIS package can simulate more than just open channel flows. It has integrated modules which can also model water quality, hydrology and sediment transport (HR Wallingford et al, 1995). The hydrodynamic part of ISIS is based on solving the St Venant equations using the Preissman 4-point implicit (box) scheme outlined earlier in this chapter. The hydraulic behaviour of the structures which can be included is governed by a combination of empirical and theoretical equations.

The program is capable of simulating looped and branched networks, and in-bank and over bank flows. Flood plains and hydraulic structures can also be incorporated. The program can be used to predict discharges and velocities, all of which are one dimensional. Supercritical flow can be included for steep channels, together with the transition between the flow states. However, the suppliers of ISIS do accept that it does not model transitions between the flow states without a smearing effect occurring, which is due to numerical diffusion. This means that that the numerical scheme that is being employed (the Preissman box scheme in this case) is unable to reproduce the rapid transition between the flow states that occurs in reality, particularly in the case of a hydraulic jump. As a result, the model tends to distribute the effect of the flow transition over a number of cross sections, instead of locating it accurately at one point.

3.3.1.1 Data input

All information manipulation in ISIS is carried out through a graphical user interface, which is referred to as the workbench. This is used to input, edit and run data, and view all results that are produced. The workbench gives access to a simulation module, network visualiser, graph manager, forms editor and tabular processor. Results are generated using the simulation module, whilst the network visualiser can be used to produce schematic plan views of the watercourse, and data can be edited in this format. Results graphs are output through the graph manager, and the forms editor provides the easiest method to edit and create new data. The tabular processor outputs long tables of results after a simulation run.

All the data necessary for a simulation run on ISIS is assembled, through the workbench, into one file. This file must contain all the river cross section data, plus a set of initial conditions for the watercourse and the boundary condition data.

Data input for the ISIS model splits the river into a series of reaches. Each reach has a defined cross section, based on surveyed levels and distances taken on site. The cross sections are split into vertical slices, and each slice can have its own roughness value based on different bed materials, bank side vegetation or other factors, as shown in figure 3.2. The distance between each cross section (the length of the reach) is taken along the river centre line, to account for bends and meanders. This is the only account which is taken of deviations from a straight path. Since the model is one dimensional, the relative orientation of sections is not included.

A wide variety of hydraulic structures can be included in ISIS. Several different types of weir can be included as part of the data file, together with many varieties of sluices, head loss inducing factors such as restrictions, plus flumes, siphons and pumps.

Flood plains can either be input into the model as part of the river cross sections, or separated from the main river and linked by a spill unit. The method chosen should best reflect the scenario which occurs in reality. If the flood plain acts as a storage area only, it may be best to incorporate it as a spill unit with an associated storage reservoir. The spill unit takes the form of a level control weir, where the weir level varies along the length of the watercourse. If flow along the flood plain contributes to the passage of water, then it should be included in the river cross sections.

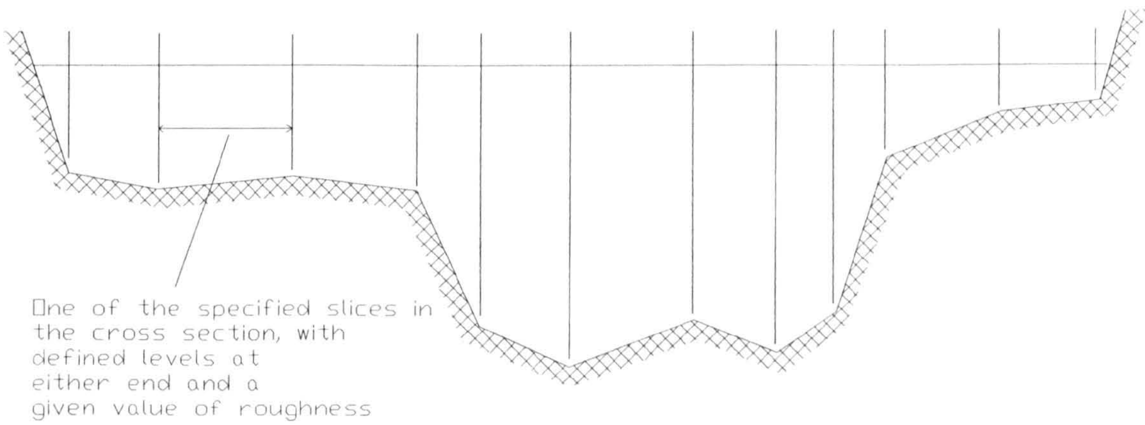


Figure 3.2. Splitting of a Typical River Cross Section into Slices for Input into the ISIS model.

3.3.1.2 Boundary and Initial Conditions

Before running a simulation on ISIS, data needs to be included on the two governing boundary conditions, and the initial conditions at the start of the simulation. This information all goes into the data file. Model boundaries in ISIS are either flow / time, stage / time, or stage / flow (also known as a rating curve). The upstream condition commonly takes the form of an inflow hydrograph, whilst the downstream condition relates discharge and depth, or depth and time.

All of the above types of boundary condition can be input by hand by the user, as a series of twin pairs of variables at a number of discrete intervals. Past flow events on the river can be simulated in this way, to test the models ability to reproduce the recorded water levels at different sections along the watercourse. As an alternative, the package includes Flood Studies Report (FSR) flow boundaries. These will either generate flow hydrographs for design return events, or will simulate runoff during historic events using recorded rainfall and related input data based on unit hydrograph theory.

Initial conditions are required to be defined as a series of values of depth and flow, specified so as to cover the entire river length under consideration.

3.3.1.3 Results

ISIS can be used to solve steady and unsteady flows. Solutions from steady state runs are often used as the initial conditions for an unsteady run. Two options are available for solving steady flows; the pseudo time stepping method and the direct method. The pseudo time stepping method uses the Preissman scheme. In this method, the boundary conditions remain at a constant value, governed by the time at which a steady state solution is being obtained. An initial guess at the solution is made, in terms of discharges and depths throughout the length of the watercourse. The Preissman scheme is then used to progress the answer through time, using the fixed boundary conditions and the initial guess at the solution values. This continues until a steady state is obtained.

The direct method uses a fourth order Runge-Kutta solver. The St. Venant equations are reduced to a set of ordinary differential equations (because the time derivative can be dropped), which can be solved for each reach. The method uses an iterative scheme to ensure that the water levels are the same at junctions between reaches. Use of the direct method holds an advantage, as the user will be told if the method has automatically added extra interpolated cross sections in order to compute a solution. If this occurs, it indicates that there is a significant change in properties between adjacent cross sections, and this argues that additional sections should be surveyed, in these locations, for input into the model.

For an unsteady run, the user specifies the time that the run is to start and finish , and the size of the time step to be employed. The start and finish times should correspond to the input hydrograph. During unsteady and pseudo time stepping runs, ISIS displays convergence information on discharge and levels, so that the user can terminate the run if the solution is seen not to be converging.

As an additional feature, ISIS contains a boundary mode method of simulation whereby the user can carry out an unsteady run, and then extract a rating curve from the results, for a specified cross section in the channel, to use as a boundary condition in a future simulation. This is useful where the user has little or no recorded data for the extreme downstream cross section of the watercourse. In this case, the boundary mode allows the user to obtain data which can be used as an approximate boundary condition.

Results can be output in tabular format as well as graphically. Plots of any of the major hydraulic parameters against time at any selected location in the watercourse can be produced. Cross sections, long sections and plan views can also be output with the predicted water level at any time in the simulation included.

3.3.2 HEC-RAS

The package Hydrologic Engineering Center's River Analysis System (HEC-RAS) was developed by the US Army Corps of Engineers and is distributed by the Water Resources Consultancy Services (US Army Corps of Engineers, 1995). It is able to calculate subcritical, supercritical and mixed flow regimes. However, the package can only compute steady flow water profiles, unlike ISIS which can perform steady and unsteady simulations. It uses the one-dimensional energy equation for the main part of the computation. The energy equation is solved iteratively from one cross section to the next to calculate the water profile. At transitions between subritical and supercritical flow, or vice versa, the energy equation is not applicable. At these points the program applies either a momentum balance equation across the transition, or some form of empirical relationship. A more detailed explanation of these two possibilities is given in the user manual. It is considered beyond the scope of this text to enter into too much detail on these points, as no such flow transitions occurred in the reaches of river modelled in this thesis.

As with ISIS, frictional resistance is included in the computation using Manning's equation. Again, each cross section is subdivided into a number of vertical slices and each slice can have its own value of Manning's n.

All manipulation of information in HEC-RAS is carried out through a Graphical User Interface (GUI). A large number of results plots can be output including: cross section profiles, rating curves and hydrographs as well as a three dimensional plot showing the computed water surface across several cross sections. All results can also be output in tabular format.

3.4. Two Dimensional Flow Modelling

It was originally planned to use SPRINT2D (Sleigh et al, 1995) to carry out the two dimensional modelling work for this thesis. However, during the completion of the modelling work, it became apparent that SPRINT2D was not dealing with roughness accurately on the banks of the channel. SPRINT2D continues to be under development to deal with this problem. Instead, the program SSIIM was used for all the two dimensional modelling work carried out.

SSIIM solves the depth averaged Navier Stokes equations using the k-ε turbulence model to solve for the eddy viscosity. The Navier Stokes equations and turbulence modelling are discussed later in this chapter in the section on three dimensional modelling.

3.4.1 SSIIM

SSIIM stands for Sediment Simulation in Intakes with Multiblock Option. The software was originally developed at the Division of Hydraulic Engineering at the Norwegian Institute of Hydrology in 1990-1991. The version used in this thesis was developed in 1994 (Olsen, 1999a).

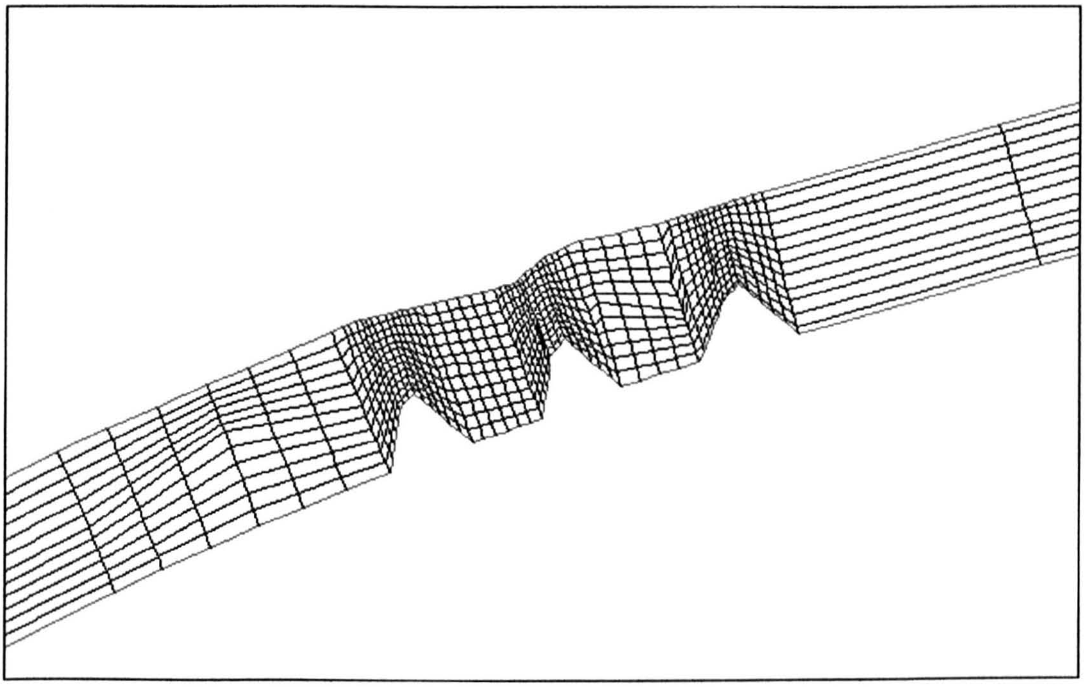


Figure 3.3. SSIIM grid for deflector 3c

The program uses a control volume method of solution on a structured non-orthogonal grid. The water surface elevation is first initialised using a standard backwater calculation based on the value of Manning's n in the input file. As the solution progresses, the water elevation at each point in the grid is updated according to the calculated pressure field. The grid is adaptive in the vertical direction to accommodate the calculated changes in depth. When the solution is seen to have converged, a steady state solution for depth and velocity is obtained at the cell centres of the grid.

The model has its own graphic user interface, which can plot velocity vectors and scalar variables. The plots show a two dimensional view of the three dimensional grid, in plan view. A typical grid employed by SSIIM to solve the flow problems detailed in Chapter 4, is shown in Figure3.3.

3.4.1.1 Problem Definition within SSIIM

The geometry of a problem in SSIIM is defined in a single file called the koordina (Olsen 1999b). This file details the locations of the vertices of all of the cells in the grid. The bed elevation at each of the cell vertices is also specified. The data appears as a set of three dimensional spatial co-ordinates, which must be in the form of a number of cross sections. The number of points across the channel cannot vary between sections.

Data on the boundary conditions and other governing factors for the simulation is contained in the control file. The upstream boundary condition is defined as a continuous discharge, and the downstream condition as a fixed water surface elevation. The boundary roughness is specified by a Mannings *m* value, which is the inverse of the Manning's *n* value. It is possible to specify both a Mannings *m* value and an effective roughness value, k_s. In this case, the Mannings *m* value is used to initialise the water surface elevation using a backwater calculation. This elevation is then recalculated throughout the period of the solution based on calculations using the k_s roughness value.

3.5. Three Dimensional Modelling

Flows within a river or watercourse are turbulent. This means that the flow is inherently unstable, and a large degree of turbulent mixing takes place as water flows downstream. In one and two dimensional models, the turbulent properties of the flow are not solved explicitly. Instead, the effects of turbulence are taken into account in the use of the DeChezy or Manning roughness coefficients. Three dimensional models solve the flow structure in much more detail, and as a result the turbulent properties of the flow can be analysed as part of the process of solving a problem. However, the analysis of turbulence is complex, and to solve turbulent flow in three dimensions is not straightforward. The phenomenon of turbulence is discussed later in this chapter.

Turbulent flow in three dimensions is described by the Reynolds averaged equations, which are in turn derived from the Navier Stokes equations. These four equations contain ten unknowns so they cannot be solved directly. A variety of computational methods have been developed so that solutions can be generated using numerical procedures.

As it was not the purpose of this thesis to give a comparison of the results obtained from the use of different turbulence models, the literature review here does not include the detailed mathematics of each of the different approaches. k-ε models are discussed in the most detail as they form the basis for a lot of the work outlined later in this thesis. Other turbulence models are mentioned for completeness only. Of these, Reynolds stress models were the only other tested in this thesis. A comparison between the results produced by a Reynolds stress model and those from the k-ε model showed very little difference in the situation that was tested. However, the Reynolds stress model took considerably longer to run. The results from this test are shown in chapter 4. As a result, it was decided to confine the approach to the use of k-ε. Many texts are available for a more detailed account of turbulence modeling, for example Versteeg and Malalasekera, 1995.

Brief consideration is also given in this section to the variety of differencing schemes available. These schemes determine how the solution at one time level is used to generate the solution at the next time level. The work carried out here included a comparison between results from hybrid and CCCT schemes. This showed little difference in the results for the two schemes, in the scenario that was tested. The results from this test are shown in chapter 4. As a result, it was decided to use the hybrid scheme throughout as this took less time for each simulation run. It was also found that it was easier to obtain convergence using hybrid differencing.

As well as including more detail on all of the above, this section also includes a discussion of how boundary roughness is included in three dimensional models, and also the various packages that are available.

3.5.1 The Navier-Stokes Equations

Fluid flow in three dimensions is governed by the Navier-Stokes equations, equation 3-5 to 3-8 (Versteeg and Malalasekera, 1995). This is a closed set of four equations with four unknowns u, v, w and p. The equations represent overall mass conservation, and momentum balance for each of the three directions.

x, y and z represent the three mutually perperdicular axis and u, v and w are components of velocity in each of these directions

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=0$$

(3-5) – mass conservation

$$\rho \frac{Du}{Dt} = -\frac{\partial p}{\partial x} + div(\mu\, grad\, u) + S_{Mx}$$

(3-6) momentum balance in x direction

$$\rho \frac{Dv}{Dt} = -\frac{\partial p}{\partial y} + div(\mu\, grad\, v) + S_{My}$$

(3-7) momentum balance in y direction

$$\rho \frac{Dw}{Dt} = -\frac{\partial p}{\partial z} + div(\mu\, grad\, w) + S_{Mz}$$

(3-8) momentum balance in z direction

The S_M terms are the momentum source terms, p is the pressure

It has been observed that below a certain value of Reynolds number (UL/ν where U and L are characteristic velocity and length scales of the mean flow and ν is the kinematic viscosity) the flow is laminar, and above that number the flow is turbulent. Laminar flows exist where adjacent layers of fluid slide past each other without significant mixing taking place. This type of flow can be completely described by the Navier-Stokes equations in the format already given. Flows in rivers are turbulent, and the analysis of this type of flow is more complex. In order to develop this argument, it is first necessary to explain what turbulence is.

Above a certain value of the Reynolds number, fluid motion becomes intrinsically unsteady. Flow properties vary randomly and chaotically. Figure 3.4 shows how velocities may vary at a point in the flow.

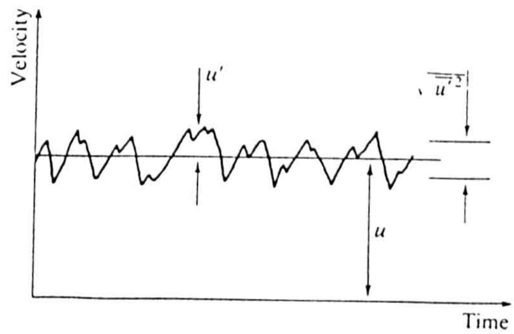


Figure 3.5 Variation of velocity with time in turbulent flow.

Figure 3.4. Variations of velocity with time in turbulent flow. Taken from Chadwick and Morfett, 1986

Turbulent flow contains rotating flow structures known as eddies. These typically exhibit a wide range of length scales. Eddying motion tends to generate very effective mixing within the flow structure. As a result, heat, mass and momentum are readily exchanged. The presence of these eddies at a wide range of length scales makes the analysis of turbulent flows problematical. Versteeg and Malalasekera reports that a flow domain of 0.1m by 0.1m with a high Reynolds number turbulent flow may contain eddies as small as 10 to 100um in size. A computing mesh of 10^9 to 10^{12} points would be necessary to resolve the flow structure to the smallest scale. In addition, the frequencies of events could be as fast as 10kHz. This would require a time step of 100us to capture every event. The computing power necessary to carry out these calculations is in the order of 10 million times faster than a CRAY supercomputer. As a result of this, it is not currently possible to track the dynamics of every eddy in a turbulent flow. However, alternative procedures have been developed which can cope with the analysis of turbulence by dealing with the time-averaged properties of the flow. Before we can do this it is necessary to incorporate the effects of turbulence into the Navier-Stokes equations.

3.5.2 Reynolds Stresses

As a result of its chaotic nature it is impossible to resolve the path of every eddy in a turbulent flow. Therefore, a time-averaged analysis of the flow is used. Each of the flow properties of the fluid is considered as a steady mean component with a time-varying fluctuating component. The mean of each of the fluctuating parts is zero. The turbulent flow velocities u(t) are replaced by a steady mean value U with a fluctuating component u'(t) superimposed on top: u(t)= U+ u'(t). Each of the flow properties is described in this way, using a mean value with a fluctuating component. The same analysis is applied to a fluctuating vector quantity so that $\mathbf{a} = \mathbf{A} + \mathbf{a}'$. Replacing each of the flow variables (i.e. the three components of velocity and the pressure) in the Navier-Stokes equations by the sum of a mean and fluctuating component and doing some rearranging leads to equations 3-9 to 3-12 which are known as the Reynolds equations.

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=0$$

(3-9)

$$\frac{\partial U}{\partial t}+div(U\mathbf{U})=-\frac{1}{\rho}\frac{\partial P}{\partial x}+div(\nu\, grad\, U)+\left[-\frac{\partial \overline{u'^2}}{\partial x}-\frac{\partial \overline{u'v'}}{\partial y}-\frac{\partial \overline{u'w'}}{\partial z}\right]$$

(3-10)

$$\frac{\partial V}{\partial t}+div(V\mathbf{U})=-\frac{1}{\rho}\frac{\partial P}{\partial y}+div(\nu\, grad\, V)+\left[-\frac{\partial \overline{u'v'}}{\partial x}-\frac{\partial \overline{v'^2}}{\partial y}-\frac{\partial \overline{v'w'}}{\partial z}\right]$$

(3-11)

$$\frac{\partial W}{\partial t} + div(W\mathbf{U}) = -\frac{1}{\rho}\frac{\partial P}{\partial z} + div(\nu\, grad\, W) + \left[-\frac{\partial \overline{u'w'}}{\partial x} - \frac{\partial \overline{v'w'}}{\partial y} - \frac{\partial \overline{w'^2}}{\partial z}\right]$$

(3-12)

It can be seen that the continuity equation is unchanged, but the three momentum equations have, between them, gained six extra terms, equation 3-13. These terms are known as the Reynolds stresses and represent convective momentum transfer within the flow, due to the presence of the velocity fluctuations. Computational procedures are required to solve the equations with these extra terms.

$$-\rho\overline{u'^2},\quad -\rho\overline{v'^2},\quad -\rho\overline{w'^2},\quad -\rho\overline{u'v'},\quad -\rho\overline{u'w'},\quad -\rho\overline{v'w'}$$

(3-13)

3.5.3 Solution of the Turbulent Flow Equations

A variety of turbulence models are available for this purpose. The first to mention is the use of large eddy simulations (LES). This method does not actually solve the Reynolds Stress Equations, but deals with the Navier Stokes equations. In this method, the equations are solved for the mean flow and the largest eddies. Effects of the smaller eddies are modelled. These methods use a great deal of computing power and are not widely used. Versteeg and Malalasekera refer to the work of Abbot and Brasco for a more in depth discussion of LES methods.

Classical methods are much more widely used than the LES approach. They are all based on obtaining solutions to the Reynolds Stress equations. The mixing length and *k-ε* models are the most popular. These two methods rely on the premise that there exists an analagy between viscous stresses and Reynolds stresses. Newton's law of viscosity provides that the viscous stresses are proportional to the rate of deformation of the fluid elements. In addition, Boussinesq proposed in 1877 that the Reynolds stresses could be linked to the mean rates of deformation. This gives rise to equation 3-14.

$$\tau_{ij} = -\rho\overline{u'_i u'_j} = \mu_t\left(\frac{\partial U_i}{\partial x_j} + \frac{\partial U_j}{\partial x_i}\right)$$

(3-14)

Equation 3-14 shows that the rate of turbulent momentum transport is proportional to the mean gradients of velocity.

A kinematic turbulent or eddy viscosity can also be defined, equation 3-15

$$\nu_t = \mu_t / \rho$$

(3-15)

Turbulence levels and turbulent stresses vary from point to point in the fluid. The value of the turbulent eddy viscosity also changes. The next stage in the discussion of alternative approaches is to outline how different approaches deal with this.

Mixing length models provide the simplest approach. They describe the stresses using a simple algebraic formula for μ_t as a function of position. *k*-ε models are more sophisticated and allow for the effects of transport of turbulent properties by the mean flow and diffusion for the production and destruction of turbulence. A transport equation for the turbulent kinetic energy *k*, and one for the rate of dissipation of turbulent kinetic energy ε are solved.

Both mixing length and *k*-ε models rely on the assumption that turbulent viscosity μ_t is isotropic. This means that the ratio between Reynolds stress and mean rate of deformation is the same in all directions. This assumption can be lead to inaccurate results in some situations. As a result, more complex Reynolds stress equation models have been developed. These models utilise transport equations for the Reynolds stresses themselves. This is theoretically possible since the Reynolds stresses arise out of the convective momentum exchanges resulting from the instantaneous velocity fluctuations. Fluid momentum is transported by fluid particles and so Reynolds stresses can also be transported within a fluid. Six extra equations are required in this approach; one for each of the Reynolds stresses. Reynolds stress models are more demanding in computing power than k-ε methods and are not as widely validated as yet. One further approach that will be discussed is the use of algebraic stress models. Here the equations describing the transport of the Reynolds stresses are algebraic expressions. These are solved alongside the equations for k and ε. This provides a relatively economical way of modelling anisotropic effects in the turbulence. The mixing length and k-ε approaches are now discussed in more detail.

3.5.4 Mixing Length Model

The mixing length model depends on a mean flow length scale l_m (m) to describe the turbulence. l_m (m) is determined as a function of its position within the flow, often by means of a simple algebraic formula. The basic formula for this approach is shown in equation 3-16. This is known as Prandtl's mixing length model.

$$\nu_t = l_m^2 \left| \frac{\partial U}{\partial y} \right|$$

(3-16)

In this approach dU/dy is taken to be the only significant mean velocity gradient. As a result the turbulent Reynolds stress is given by equation 3-17.

$$\tau_{xy} = \tau_{yx} = -\rho \overline{u'v'} = \rho l_m \left| \frac{\partial U}{\partial y} \right| \frac{\partial U}{\partial y}$$

(3-17)

Mixing length models are easy to implement and do not require a great deal of computing power. However, they cannot describe flow separation and recirculation.

3.5.4.1 The k-ε model

The next step up from the mixing length model is the k-ε approach. This method focuses on the dynamics of turbulence by considering the mechanisms that affect the instantaneous turbulent kinetic energy, $k(t)$. Instantaneous turbulent kinetic energy is defined by equation 3-18, 3-19 and 3-20.

$$K = \frac{1}{2}\left(U^2 + V^2 + W^2\right)$$

(3-18) K is the mean kinetic energy

$$k = \frac{1}{2}\left(\overline{u'^2} + \overline{v'^2} + \overline{w'^2}\right)$$

(3-19) k is the turbulent kinetic energy

$$k(t) = K + k$$

(3-20) for the instantaneous kinetic energy

This method also utilises the concept of a dissipation rate of turbulent kinetic energy, ε. This dissipation arises out of the work done by the smallest eddies against the viscous forces. k and ε are used to define a velocity scale, ϑ, and a length scale, ι, as shown in equation 3-21 and 3-22. These scales are related to the large scale eddies within the flow.

$$\vartheta = k^{\frac{1}{2}}$$

(3-21)

$$\iota = \frac{k^{3/2}}{\varepsilon}$$

(3-22)

It should be noted that the value of ε, the dissipation rate of turbulent kinetic energy, has been used in the definition of the large eddy scale ι This appears to be an anomaly at first as the value of ε is related to the work done by the smaller eddies. However, this is possible since at high Reynolds numbers the rate at which large eddies extract energy from the mean flow is exactly balanced by the transfer of energy to the smaller, energy dissipating eddies. This has to be the case otherwise energy would grow or diminish without end at one of the scales of turbulence. As a result ε can be used to define ι.

It is now possible to define the eddy viscosity in terms of the quantities we have specified. This is shown in equation 3-23.

$$\mu_t = C\rho\vartheta\iota = \rho C_\mu \frac{k^2}{\varepsilon}$$

(3-23) where C_μ is a dimensionless constant.

The standard model uses two equations, one for k and one for ε. These are shown in equation 3-24 and 3-25.

$$\frac{\partial(\rho k)}{\partial t}+div(\rho k\mathrm{U})=div\left[\frac{\mu_t}{\sigma_k}grad\,k\right]+2\mu_t E_{ij}.E_{ij}-\rho\varepsilon$$

(3-24)

$$\frac{\partial(\rho\varepsilon)}{\partial t}+div(\rho\varepsilon\mathrm{U})=div\left[\frac{\mu_t}{\sigma_\varepsilon}grad\,\varepsilon\right]+C_{1\varepsilon}\frac{\varepsilon}{k}2\mu_t E_{ij}.E_{ij}-C_{2\varepsilon}\rho\frac{\varepsilon^2}{k}$$

(3-25)

This can be expressed in words as:

Rate of change of k or ε	+ Transport of k or ε by convection	= Transport of k or ε by diffusion	+ Rate of production of k or ε	-	Rate of destruction of k or ε

The model uses five constants C_μ, σ_k, σ_ε, $C_{1\varepsilon}$, $C_{2\varepsilon}$. Standard values for these constants have been determined through extensive data fitting procedures for a wide range of flows. These values are:

$$C_\mu=0.09;\ \sigma_k=1.00;\ \sigma_\varepsilon=1.30;\ C_{1\varepsilon}=1.44;\ C_{2\varepsilon}=1.92$$

Production and destruction of turbulent kinetic energy are closely linked. As a result, when the value of ε, the dissipation rate, is large the production of k is large. The production and destruction terms of equation 3-24 are proportional to those of equation 3-25. Thus if production of one term is high, the other will be correspondingly high. This avoids negative values of turbulent kinetic energy being computed.

k-ε is the most widely used and validated of all the turbulence models. It has been widely successful in predicting flows in a wide variety of circumstances without the need to alter the model constants. The method has been used to simulate flows in rivers. Results showed that the method was able to predict the main feature of the flow to the level of accuracy required (Cokljat and Kralj, 1997). However, the model has been found to produce inaccuracies in some cases, particularly in dealing with swirling flows, some unconfined flows, rotating flows or flows with large extra strains.

3.5.5 Numerical Methods

As was the case with one dimensional modelling, a number of different techniques are available to solve the flow equations. Finite difference methods have already been discussed in the earlier section on one-dimensional modelling. An alternative is the use of finite element methods. These use simple functions that are valid along elements to describe the variations of the unknown variables.

Another option is finite volume methods, which require the governing equations to be integrated over finite control volumes of the solution domain. Discretisation then follows whereby finite difference type approximations are used to express the terms in the integrated form of the governing equations. This converts the integrals into a system of algebraic equations which are solved iteratively.

3.5.5.1 Differencing Schemes

The governing equations of three dimensional fluid flow are a set of partial differential equations. In order to be able to solve these equations using computers, they must be converted to what is known as a numerical analogue. This consists of discrete values at nodes within the solution domain. The process of converting the partial differential equations is known as discretisation. The process of discretisation takes a different form when dealing with either a finite difference, finite volume of finite element method of solution. Since CFX employs a finite volume method of solution, the following discussion will be limited to discretisation methods for that approach.

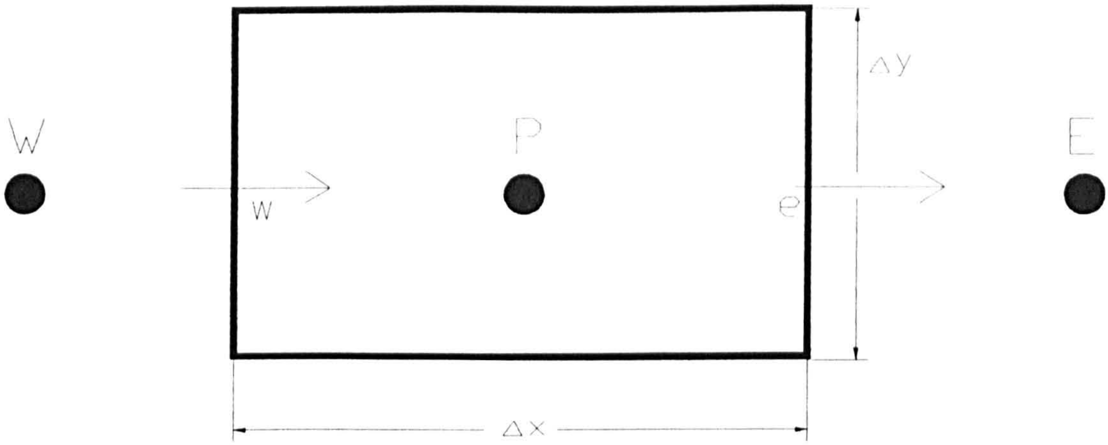


Figure 3.5 A finite volume.

The finite volume method involves solving the flow equations for small elements within the fluid, for example that shown in Figure 3.5. If we were considering a general variable ϕ for which the first derivitive is shown in equation 3-26.

$$\frac{\partial \phi}{\partial x}$$

(3-26)

The control volume integration of equation 3-26 yields equation 3-27.

$$\int_{\Delta V} \frac{\partial \phi}{\partial x} dV = (\phi_e - \phi_w)\Delta y$$

(3-27)

Integration of the governing equations reveals that it is necessary to determine values at the faces of the control volume. This can be done by linear interpolation as shown in equation 3-28.

$$\phi_e = \frac{\phi_P + \phi_E}{2}, \qquad \phi_w = \frac{\phi_W + \phi_P}{2}$$

(3-28)

Substituting equation 3-28 into equation 3-27 yields equation 3-29

$$\int_{\Delta V} \frac{\partial \phi}{\partial x} dV = \left[\frac{\phi_P + \phi_E}{2} - \frac{\phi_W + \phi_P}{2} \right] \Delta y$$

$$= \left(\frac{\phi_E - \phi_W}{2} \right) \Delta y$$

(3-29)

In finite difference form equation 3-29 can be represented as shown in equation 3-30

$$\frac{\partial \phi}{\partial x} = \frac{\phi_E - \phi_W}{2 \Delta x}$$

(3-30)

Equation 3-30 is the "central" differencing scheme. This scheme works well for slower flows but becomes unstable for cell Reynolds numbers greater than 2. An alternative approach is known as upwinding. This interpolation takes account of the flow direction when determining the values of ϕ to take at the cell faces as shown in equation 3-31.

$$if \;\; u \geq 0 \;\; \phi_e = \phi_P$$
$$if \;\; u < 0 \;\; \phi_e = \phi_E$$

(3-31)

Equation 3-31 leads to equation 3-32 for the discretisation over the control volume

$$\frac{\partial \phi}{\partial x} = \frac{\phi_P - \phi_W}{\Delta x} \quad if\ u \geq 0$$

$$\frac{\partial \phi}{\partial x} = \frac{\phi_E - \phi_P}{\Delta x} \quad if\ u < 0$$

(3-32)

The upwind scheme is very stable. A further improvement is the hybrid scheme, which combines the central and upwind techniques. Central differencing is used for slow flow regions and upwinding for fast flow regions. Hybrid differencing was the method used in all the three dimensional work presented in this thesis. Results are presented in chapter 4 that show that the results from this discretisation are extremely similar to those from a more complex differencing technique.

Many more complex differencing techniques are available. Common ones include CCCT, CONDIF, SHARP and QUICK. These schemes use values at more neighbouring points, and hence a wider range of influence, in determining the values at the cell faces of the control volume. A detailed discussion of these more complex differencing techniques is beyond the scope of this text.

3.5.5.2 Wall Boundary Conditions

It is necessary to consider how to represent the effects of wall friction in a three dimensional model. In a one dimensional model, roughness can be included using one empirical coefficient. Typically, either the Chezy or Manning coefficient is used for this purpose. In the three dimensional case, the situation is more complex and it is necessary to consider what happens to the turbulence in the near wall region where the flow is dominated by viscous effects. Generally it is not possible to resolve the flow field all the way down to the wall boundary as this would require an excessively large number of grid points. Instead, formulae are employed to specify the flow conditions immediately adjacent to the solid boundary. These formulae are termed wall functions

Typically in the analysis of the near wall flow field, two dimensionless quantities are specified. These are u^+ and y^+ which are dimensionless quantities related to the flow velocity and the distance from the wall respectively. u^+ and y^+ are defined in equation 3-33 which is known as the law of the wall. Different modelling packages may specify u^+ and y^+ differently.

$$u^+ = \frac{U}{u_\tau} = f\left(\frac{\rho u_\tau y}{\mu}\right) = f\left(y^+\right)$$

(3-33) $u_\tau = (\tau_w/\rho)$

Fluid immediately adjacent to the wall is stationary. A very thin layer develops near the wall where viscous shear forces dominate and it can be assumed that the shear stress is approximately constant and equal to the wall shear stress τ_w. This region is known as the viscous sublayer. The shear stress in this region can be defined as shown in equation 3-34.

$$\tau(y) = \mu \frac{\partial U}{\partial y} \cong \tau_w$$

(3-34)

This leads to a linear relation between mean velocity and distance from the wall, hence equation 3-35.

$$U = \frac{\tau_w y}{\mu}$$

(3-35)

This leads to equation 3-36 for the viscous sublayer

$$u^+ = y^+$$

(3-36)

Outside the viscous sublayer a further region exists where viscous and turbulent effects are both important. This is known as the log-law layer, and the relationship between u^+ and y^+ is defined in equation 3-37. The value of κ is 0.4 and C is 5.5(E=9.8) for a smooth wall. C tends to be reduced by the effect of wall roughness

$$u^+ = \frac{1}{\kappa}\ln y^+ + C = \frac{1}{\kappa}\ln(Ey^+)$$

(3-37)

The viscous sub layer and logarithmic regions of a typical near wall velocity profile are shown in figure 3.6. A logarithmic scale is used on the horizontal axis.

One further region exists which is known as the outer layer. Here, velocities are influenced by the wall shear stress but not by viscosity itself. The velocity distribution is given by equation 3-38.

$$\frac{U_{max} - U}{u_\tau} = \frac{1}{\kappa}\ln\left(\frac{y}{\sigma}\right) + A$$

(3-38) A is a constant, σ is the thickness of the boundary layer

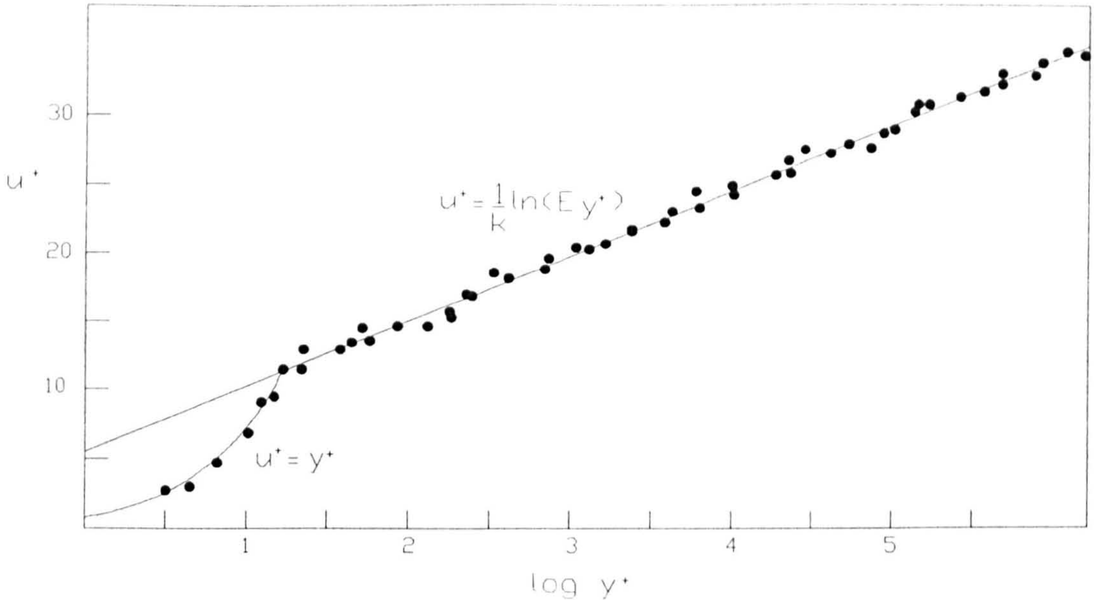


Figure 3.6 Typical near wall velocity profile plotted on a logarithmic scale. Adapted from Versteeg and Malalasekera, 1995.

3.6. Three Dimensional Modelling Packages

3.6.1 CFX

The package that was used for all of the three dimensional modelling carried out in this thesis was CFX. This product was developed by AEA Technology plc and uses a finite-volume method of solution (CFX 4, 1994). To simulate a problem, the package is used in three parts: the pre-processor, solver and post-processor.

The pre-processor in CFX is called Build and is used to: define the geometry of the problem, subdivide this region into grid cells, select the physical phenomena to be modelled, define the fluid properties and specify the boundary conditions. Geometry can be input interactively using the mouse or read from a file. Boundary conditions are referred to as patches, and can include multiple inlets. The end result of using the pre-processor is the generation of two files that are both required to run the solver, these are a geometry and a command file. The geometry file contains the geometry of the problem together with the mesh information and the position and type of the boundaries. The command file contains the fluid properties, simulation control parameters (number of iterations etc) and the details of the boundary conditions (inlet velocities etc).

The solver (CFX-Flow) uses the information supplied by the pre-processor and solves the governing equations to produce the solution. A large number of turbulence models are available within CFX. As has already been stated, this thesis focused on the use of k-ε methods only. User Fortran can be included to help in the setting up of a problem or to generate additional output. The solver allows interactive plotting of residuals and spot values, so that the user is able to follow the convergence of a run.

There are two CFX post processors called VIEW and VISUALISE. This is where the graphics are produced. Many different types of plot are available including domain geometry (with or without the grid), vector or shaded plots of any of the hydraulic variables or individual particle tracks within the fluid.

3.6.1.1 Modelling Channel Roughness in CFX

A general review of the means by which boundary roughness is incorporated into three dimensional models, using wall functions, has been given above. However, there appears to be no industry standard approach for the actual mechanism by which this is carried out. This is probably due to the fact that three dimensional modelling is not, as yet, a widely tool in analysing river flows. Within CFX the boundary roughness can be altered, as required, by modifying two values in the command file. These values influence the near wall velocity profile. The modified values are the dimensionless loglayer constant (E') and sublayer thickness (y^+). Formulae for modifying the value of E' and y^+ based on the value of the effective grain roughness k_s have been suggested by Hodkinson (1996) and this procedure is followed in all the three dimensional modelling carried out in this thesis. In outlining this method, Hodkinson makes reference to earlier work on channel friction by Knighton, and on the relation between effective roughness heights and grain size of Bray and Ferguson. The loglayer constant E' is first defined by equation 3-39:

$$E' = \frac{E}{1 + 0.3k_s^+}$$

(3-39) E is the value for a smooth wall (9.8) and k_s^+ is defined by equation 3-40:

$$k_s^+ = \frac{\rho k_s u_*}{\mu}$$

(3-40) Where ρ is the fluid density, and μ is the dynamic viscosity. k_s is the effective roughness height and is related to the bed grain size. A number of empirical formulae are available for the determination of k_s. van Rijn (1982) reported a range varying from $k_s = d_{90}$ to $k_s = 10d_{90}$. The relation used here is that used by Hodkinson (1996) as defined by equation 3-41. The shear velocity u_* is defined by equation 3-42.

$$k_s = 3.5D_{84}$$

(3-41) Relationship between the effective roughness and the diameter of the sediment grains on the bed.

$$u_* = gRS_0$$

(3-42) R is the hydraulic radius and S_0 is the channel slope.

Equation 3-42 assumes a uniform flow condition. This is necessary so that an average value of the shear velocity can be calculated and used in the determination of E' prior to the commencement of the cfd computation. The assumption is, in general terms, reasonable as the constant channelized cross section of the Idle, together with the consistent bed slope, should permit the flow to become at or close to uniform. However, in the vicinity of the deflectors the flow will not be uniform and so the relation of equation 3-42 does not hold. No alternate method for the determination of the shear velocity, prior to the start of the flow simulation, exists. The next version of CFX to be released will allow the user to input a value for the effective grain roughness into the command file. The program will then automatically calculate modified values of the log layer constant and sub layer thickness at the end of each iteration for each point in the solution domain, using the calculated value of the shear velocity at that point (Hargreaves, 1999).

The above equations allow the value of the loglayer constant (E') to be determined according to the diameter of the sediment grains in the river, the hydraulic radius and the channel slope. The procedure for incorporating the modified value into the three dimensional model to represent the boundary roughness uses the concept of wall functions introduced earlier in this chapter. Firstly the wall shear stress (τ_w), is defined by equation 3-43:

$$\tau_w^{\ 2} = C_\mu \rho^2 k^2$$

(3-43) for the wall shear stress. C_μ is a constant and k is the turbulence kinetic energy.

A new quantity τ_k is defined by equation 3-44.

$$\tau_k = \rho C_\mu^{1/2} k$$

(3-44)

The new quantity is used to define scaled variables, equations 3-45 and 3-46.

$$u^+ = \frac{(\rho \tau_k)^{1/2}}{\tau_w} u$$

(3-45)

$$y^{+}=\frac{(\rho\tau_{k})^{1/2}}{\mu}(H-y)$$

(3-46)

The scaled velocity component parallel to the wall and in the x direction is given by Equation 3-47 when $y^{+}<y_0^{+}$ and by equation 3-48 when $y^{+}>y_0^{+}$. y_0^{+} is the cross over point between the viscous sub-layer and the logarithmic region and is defined by the upper root of equation 3-49.

$$u^{+}=y^{+}$$

(3-47)

$$u^{+}=\frac{1}{\kappa}\ln(Ey^{+})$$

(3-48) κ is the von Karman constant.

$$y_0^{+}=\frac{1}{\kappa}\ln(Ey_0^{+})$$

(3-49)

By following the above procedure, modified values of E and y^+ can be calculated for each problem and put into the CFX command file to accurately represent the surface roughness. It should be noted that the values of E and y^+ that are input do not, by themselves, determine the velocity profile at the channel boundary. This is also influenced by the near wall turbulence as demonstrated by equations 3-43 and 3-44.

3.6.2 Other CFD Packages

A number of other three dimensional flow modelling packages exist. A full discussion of each of these packages is beyond the scope of this thesis. It is sufficient to say that each package models the flow in a different way. Some are better suited to modelling the position of the free surface than CFX. Others are 'layered' models. These do not solve a momentum equation in the vertical direction, but instead split the flow into vertical layers. Two, mutually perpendicular, momentum equations are then solved within each layer of the flow. In effect, this creates a number of two dimensional models on top of each other in the solution domain.

The results shown in chapter 4 indicate that the two dimensional model SSIIM does not accurately predict the flow pattern at the deflectors. SSIIM utilises a hydrostatic pressure distribution, which is also used by the layered three dimensional models. It is believed that this may be part of the reason why SSIIM is unable to predict the complex flow pattern at the deflectors accurately. As a result, it was decided to use CFX, instead of a layered model, for all the three dimensional work in this thesis, as that solves the flow field in more detail. It does this by solving a momentum equation in the vertical plane. The results shown in chapter 4 show that CFX is able to correctly predict the flow pattern at the deflectors.

3.6.3 Application of Three Dimensional Models to Natural Rivers

The application of three dimensional models to natural watercourses is still in its infancy. This is due to the difficulty involved in calculating the flow structure in three dimensions with complex bathymetry and arbitrarily shaped flow boundaries. The vast majority of existing three dimensional methods to date have focused on the study of simplified river-like geometry's. For example, the studies of Leschziner and Rodi (1979) and Demuren and Rodi (1986) used the k-ε turbulence model to simulate flows through a 180 degree bend.

More recently, Meselhe et al (1995) used the k-ε turbulence model to simulate flows through a meandering channel with a simplified trapezoidal cross section. Demuren (1993) was the first to use a more complex natural-like cross sectional shape in a three dimensional model. The geometry included some of the features typically found in natural rivers, such as longitudinal curvature and varying bed topography. A finite volume method of solution was employed in order to solve the k-ε turbulence model. Demuren's predicted velocities demonstrated reasonable agreement with site measurements.

Hodkinson (1996) used the FLUENT code to solve the three dimensional flow structure for a 90 degree bend on the River Dean, Cheshire. FLUENT uses a finite volume method of solution A simplified representation of the river cross section was used. The computational grid consisted of 75 cells along the channel, 8 cells through the depth and 30 cells across the channel. The RNG k-ε turbulence model was also used in this case. Wall roughness was represented using wall functions. A fixed lid assumption was used for the water surface with no cross channel slope. Tests using the correct, surveyed, position of the water surface showed there was very little effect on predicted velocities. The results from this study showed good qualitative agreement with site measured velocities, as the CFD code predicted the main features of the flow very accurately. Discrepancies existed in the qualitative comparison of velocities. This may be due to the fact that the model would not accept one of the required coefficients for bed roughness. The model would only run with a smoother bed specified.

The first three dimensional model to attempt to model natural river geometry in full was reported by Olsen and Stokseth (1995). A short reach (20 x 80m) of the Skona River in Norway was modelled. A very coarse mesh was used to discretise the solution domain (600 nodes). The discrepancy between velocities predicted by the model and those measured on site was found to be between 5 and 100%.

In a more recent study (Shinha et al, 1998) a three dimensional model was developed and validated for a 4km stretch of the Columbia River downstream of the Wanapum Dam. The reach includes rapidly varying topography, area contractions and expansions and the presence of multiple islands The complex geometry was simulated in full, again making use of the k-ε turbulence model. Boundary roughness was simulated using wall functions. A fixed lid was used to govern the position of the water surface. The position of the lid was determined from field and laboratory measurements from a 1:100 scale model.

The three dimensional model was calibrated against site measured velocities taken at a number of discharges. Extra roughness was added into localised areas of the model, as necessary, to improve agreement with the site data. Good agreement was then obtained between the predicted and site measured velocities at a number of different discharges.

In another study using a three dimensional model (Wu et al, 1997), the position of the water surface was successfully calculated as part of the solution procedure using the two dimensional depth averaged momentum equations. The calculated velocities from the three dimensional solution were used to derive depth averaged values. These were fed into the two dimensional equations to calculate the position of the water surface. An adaptive grid was used to accommodate the varying water surface elevation, together with a finite volume method of solution. Wall functions were used to represent boundary friction. The value of the roughness parameter E was altered according to the value of the grain roughness k_s in a similar way to that outlined above for CFX. The code used the k-ε turbulence model.

The three dimensional model was used to predict flows in a 180 degree rectangular channel bend and a 12km stretch of the River Rhine. The grid for the channel bend problem contained 56 x 28 x 20 cells. The River Rhine problem contained a 90degree bend with point bars, a submerged island and local protrusions from the river bank. A 143 x 31 x 22 grid was used in this case. In both cases the calculated water levels, main and secondary flows as well as important flow features were in reasonably good agreement with measured values.

Chapter 4

River Idle Flow Modelling

4.1. Introduction

At the design stage of any rehabilitation scheme, the first analysis required is to examine the effect of the proposals on water levels and flow patterns. The effect of the installation of the flow deflectors in the River Idle on flood levels had already been assessed in detail by the Environment Agency, prior to this thesis. As a result, it was not necessary to repeat that work here. Instead, the work outlined here is centred on very detailed modelling of individual deflector sites. This is discussed in more detail below.

This part of the thesis is a necessary forerunner of both the aquatic habitat analysis of chapter 5, and the prediction of the movement of sediments of chapter 6. The results given in both of those chapters are based on the predictions of depths and velocities that are detailed here. In this chapter the effect of the installation of the deflectors in the River Idle on the resulting depths, velocities and discharges is modelled using modern computing packages. Four pieces of software are used for this purpose: ISIS, HEC-RAS, SSIIM and CFX. All of these pieces of software are discussed in chapter 3. An outline of the modelling procedure carried out with each package is given, together with the results obtained. An analysis of the results, and a discussion of the usefulness of each of the packages in modelling river rehabilitation proposals is included later in the chapter.

Prior to the commencement of this thesis, the Environment Agency had already carried out an extensive assessment of the effect of all of the rehabilitation proposals for the Idle on water levels and flooding potential (NRA, 1995). This study used a HEC-RAS model for the entire River Idle. The results produced include for the effect of the planting of reed beds and the realignment of some banks to form cliffs, as well as for the installation of the deflectors. The study conducted by the Environment Agency indicates that the rehabilitation will not cause a significant worsening of flooding risk. A range of discharges were modelled and the largest increase in water elevation was found to be 0.13m. With normal levels of maintenance, the largest increase in water elevation would be reduced to 0.05m. This increase in flood level is small and does not compromise flood defence in the upstream settlement of Ease Retford. The Environment Agency provided the HEC-RAS data file for the whole of the Idle for this thesis.

The verification of the HEC-RAS model, constructed by the NRA for the study of the Idle, identified the most suitable value for Manning's n to be 0.085 prior to the installation of the deflectors. The Mannings roughness for a minor stream with water surface width at flood stage less than 100ft and with very weedy reaches and deep pools is between 0.075 and 0.150, with a normal value of 0.1 (Chow, 1959). Since the Idle did not have any deep pools prior to the installation of the deflectors, the value of Manning's n suggested by the NRA appears to be a little high. However, as the NRA found it necessary to use a value of 0.085 to verify their model for the whole river, this value has been adopted for all of the hydraulic modelling in this thesis.

Since the Environment Agency had already modelled the entire effect of the rehabilitation on water levels at a wide range of discharges, and this effect was found to be very small in each case, it was not felt necessary to repeat the exercise using ISIS. To recreate the entire HEC-RAS model of the Idle in ISIS, in order to compare results from the two, was considered to be beyond the scope of this study. At the commencement of this thesis, additional cross sections (surveyed by the School of Geography at the University of Nottingham) were only available for section six and deflectors 3f and 3c. Section six incorporates five deflectors (6a to 6e). Detailed ISIS and HEC-RAS models were created for the whole of section six. These models contained a mixture of the new cross sections surveyed by the School of Geography at the University of Nottingham, and those from the original HEC-RAS model. Extra sections were included around the deflectors, as necessary, to ensure their geometry was accurately represented. A comparison of the predicted water levels for the two packages is given later in this chapter.

In addition to the ISIS and HEC-RAS models of section six, individual ISIS, HEC-RAS, SSIIM and CFX models were created for deflector sites 6a, 3c and 3f as shown in figure 4.1. The predicted velocities and depths from the various packages, for these three sites, provides the information used in both the habitat and sediment modelling of later chapters. The models of these short reaches incorporate the data provided by the Environment Agency together with the additional, more recently surveyed, cross sections. The sections surveyed by the School of Geography were taken at five or ten metre intervals upstream and downstream of each of the deflectors. Ten measurements of the bed elevation were taken per cross section, except at deflector 3f where twenty measurements were taken at each cross section.

It is incorrect to isolate sites and reaches from the one dimensional model of the entire river in the way outlined, since there may be backwater effects from rehabilitation measures downstream that may, in reality, affect the sites that are under consideration. However, the Environment Agency's HEC-RAS model has already demonstrated that the overall backwater effect of the deflectors is small (a few centimetres increase in water level). For the purpose of comparing HEC-RAS and ISIS results, and to demonstrate the output that can be achieved using these packages, the approach of focusing on individual deflector sites was considered to be the most appropriate. In addition, the excessive run times and the size of results files produced with CFX meant that only fairly short reaches could be modelled with the required level of accuracy. ISIS results for the flow depth for sites 3c, 3f and 6a (the most upstream deflector in section six) were used to fix the position of the free water surface in the three dimensional model. The reasons for this are discussed later.

4.2. One Dimensional Flow Modelling

Four simulations, each with a different discharge, were carried out using ISIS for deflector sites 3f, 3c and 6a. The results from the one, two and three dimensional modelling at these locations and discharges are the subject of the habitat and sediment modelling presented in chapters 5 and 6. Discharges selected were $8m^3/s$, $0.59m^3/s$ and $0.3m^3/s$. The fourth discharge was that which was present when each of the deflector sites were surveyed, and this was in the range of 1.204 to1.637m^3/s. For deflectors 3f and 3c the 'initial' discharge was that present in the first survey after the deflectors were installed. For deflector 6a the 'initial' flow was that present in the survey prior to the deflectors being installed. These four discharges are subsequently referred to, in order of magnitude highest first, as 'high', 'initial', 'low' and 'extra low'. The discharges simulated are shown in table 4.1.

	Deflector site 6A	Deflector site 3C	Deflector site 3F
High Flow (m^3/s)	8		
Initial Flow (m^3/s)	1.637	1.204	1.252
Low Flow (m^3/s)	0.59	0.59	0.59
Extreme Low Flow (m^3/s)	0.3	0.3	0.3

Table 4.1. Discharges simulated at the three deflector sites.

Each discharge produces subcritical flow. The discharge of $8m^3/s$ represents a high value for the River Idle. Based on flow metering data supplied from the Mattersey gauging station from the period 01/01/1995 to 01/08/1996 this discharge is only exceeded 2% of the time. Based on summer gauging data between June and September 1995, a discharge of $0.59m^3/s$ is exceeded 50% of the time, and $0.3m^3/s$ is exceeded 90% of the time. Thus, one flood discharge is being used together with two summer low flows. This should give an indication of the effect of the deflectors at extreme flows.

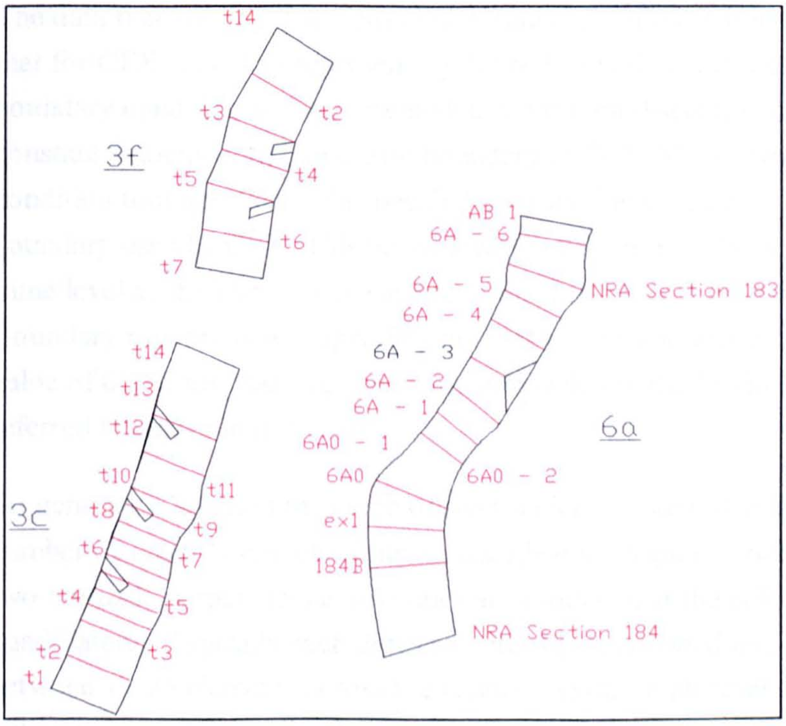


Figure 4.1. Deflector sites 3c, 3f and 6a. The flow direction is upwards in each case.

Extreme high discharges are particularly important in terms of the effect of the deflectors on flood peak levels. Very low discharges are most significant in terms of aquatic habitat. At low flows fish need deeper pools for shelter, thus flow variability is particularly important. The movement of sediments is greater at high flows, but they occur correspondingly less frequently. Thus medium and smaller flows can be equally important as they occur more often.

Results from ISIS and HEC-RAS for section six were compared at a discharge of $8m^3/s$. The largest discrepancy between the two sets of predicted water surface elevations was only 0.01m indicating good agreement between the two programs.

ISIS has the capability to model the effect of an entire flow hydrograph. However, in this thesis only single values of discharge were used as the upstream boundary condition since both HEC-RAS and CFX (in the mode used here) cannot deal with time varying problems where the position of the free surface alters during the period of the computation.

4.3. Two Dimensional Flow Modelling

The two dimensional flow modelling for this thesis was carried out using the SSIIM which has previously been discussed in chapter 3. Unlike CFX, SSIIM is capable of computing the location of the free water surface.

The data that was input to SSIIM to create each of the problems was very similar to that for CFX. The data representing the bed elevation was the same. The upstream boundary condition was represented as a constant discharge, as opposed to the constant velocity at the upstream boundary in CFX. The downstream boundary condition took the form of a specified constant head elevation instead of the mass flow boundary used by CFX. This head elevation was derived from ISIS and was at the same level as the fixed water surface at the downstream boundary used by CFX. Boundary roughness was specified in exactly the same way as ISIS; using a constant value of 0.085 for Manning's *n* as recommended in the Environment Agency report referred to earlier in this chapter.

To generate a solution the reach of river under consideration is subdivided into a number of quadrilateral elements as described in chapter 3. Solutions for depth and two mutually perpendicular velocities are produced at the centroid of each quadrilateral. Typically each deflector site was subdivided into 1260 quadrilaterals with between 13-25 elements across the channel width. In all cases SSIIM was seen to be failing to correctly predict the flow pattern in the vicinity of the deflector (see results later in this chapter). As a result, grid independence was checked for by doubling the number of quadrilateral elements in the solution domain (plot 4.20). This had no effect on the solution produced. Results for this grid independence check are shown later in the chapter.

SSIIM moves towards a solution by solving the governing equations over the domain under consideration in an iterative process. The final result file is only output when the run has converged. Convergence information for velocity, continuity and turbulence parameters is displayed on the screen as the computation of the solution progresses.

As the computation of the solution progresses, plots of velocity, water surface elevation and depth can be obtained. The data can be represented on cross sections, long sections or in plan view. In addition, flow speed arrows can be produced which help to visualise the flow pattern and are especially useful in the vicinity of the deflector. On the completion of a simulation, SSIIM can write an ASCII text file. Unfortunately, this is not in a very usable format. As a result, a separate FORTRAN program was written as part of this thesis to convert the text output file generated by SSIIM into usable data.

It was considered to be sufficient to perform SSIIM simulations only at the 'initial' discharge for each of the three deflector sites. This enabled validation of the results against site measured velocities to be carried out.

4.4. Three Dimensional Flow Modelling

The three dimensional flow modelling for this thesis was carried out using the package CFX. This software is discussed in chapter 3. This software does not allow the use of a discharge boundary condition, so the discharge was divided by the cross section area of flow at the upstream boundary in order to specify an incoming velocity. This provides a uniform velocity over the entire upstream boundary. This is not a true representation of the flow field as, for example, velocities would naturally tend to be faster near the surface and slower near the bed. To allow the flow field to develop, extra straight lengths of river were added in the model upstream and downstream of the reach where the deflector was located. The downstream boundary condition was a simple mass flow boundary.

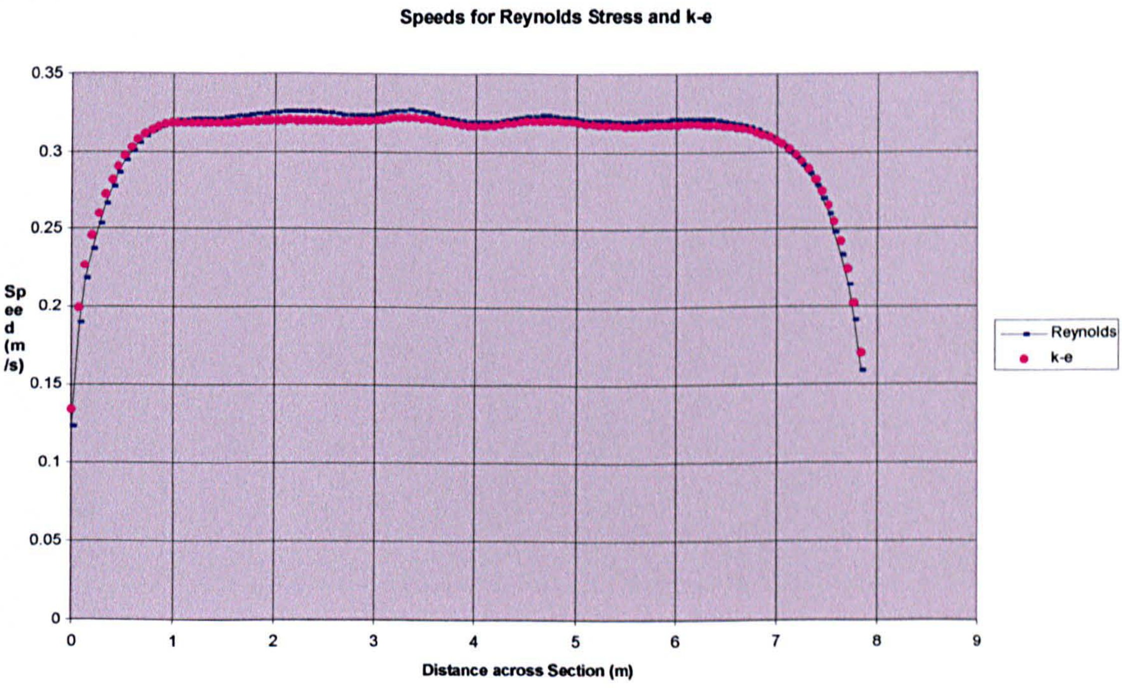


Figure 4.2 – Comparison of speeds produced using *k*-ε and Reynolds Stress models at section 1

The k-ε turbulence model was used throughout in the three dimensional modelling as initial tests on a reach of the Idle showed that the velocities it predicted were sufficiently similar to those produced using the Reynolds Stress model. Results from these tests are shown in figures 4.2 and 4.3. These figures were produced using the CFX post processor VISUALISE. This allows speeds to be plotted along any specified line which passes through the solution domain. Figures 4.2 and 4.3 were produced at two different depths and cross sections at deflector site 3f. The similarity between the two sets of results is evident in the plots.

k-ε and Reynolds Stress models are discussed in more detail in chapter 3. The k-ε turbulence model simulations took substantially less time to run than those using the Reynolds Stress model, and it was found to converge more readily.

A single simulation run was also carried out without any turbulence model (i.e. with a laminar three dimensional flow model). The purpose of doing this was to examine how significant turbulence was within the flow. Results from this exercise are presented later in this chapter.

The hybrid differencing scheme was also used in preference to CCCT as the results produced were similar, but run times were less with the hybrid scheme. Hybrid and CCCT differencing are also discussed in chapter 3. A comparison of results using CCCT and the hybrid scheme are shown in figures 4.4 and 4.5. Again, two different line graphs have been produced at different depths and cross sections for deflector site 3f.

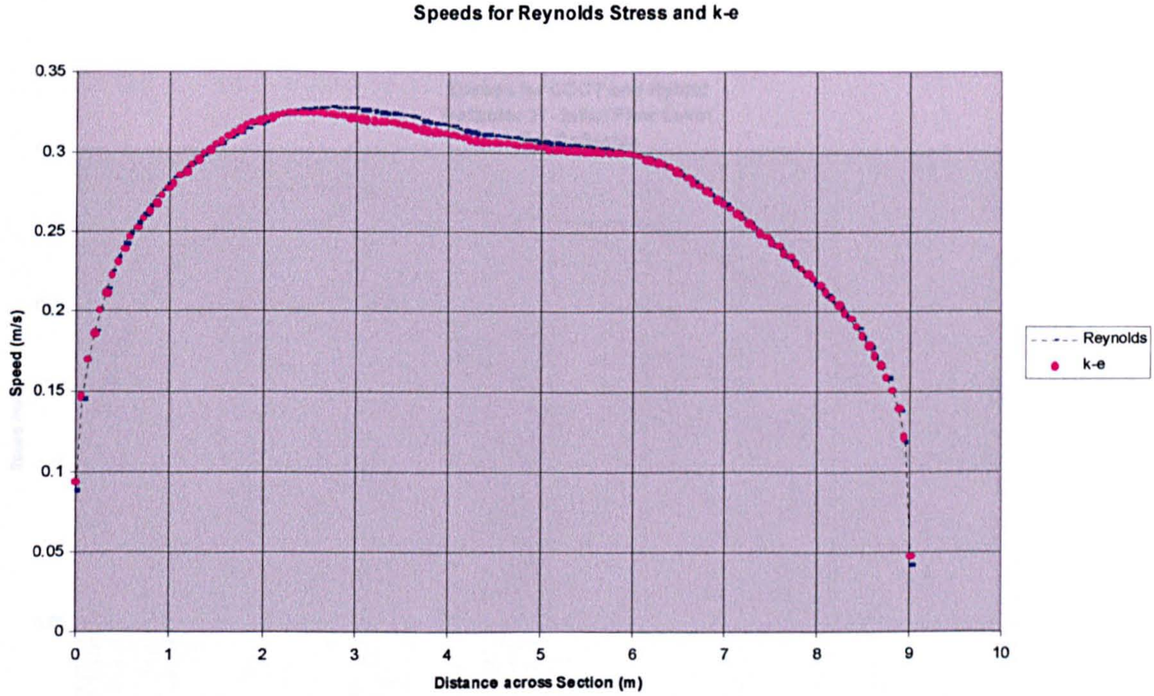


Figure 4.3 - Comparison of speeds produced using k-ε and Reynolds Stress models at section 2

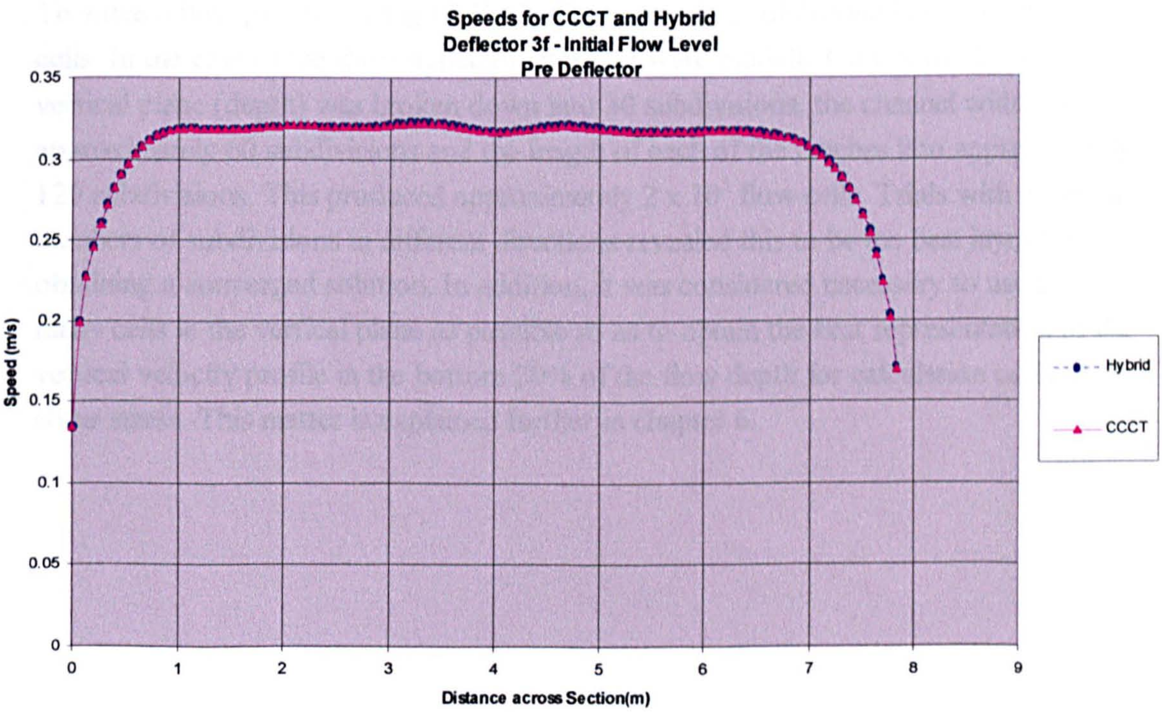


Figure 4.4 - Comparison of speeds produced using Hybrid and CCCT differencing at section 3

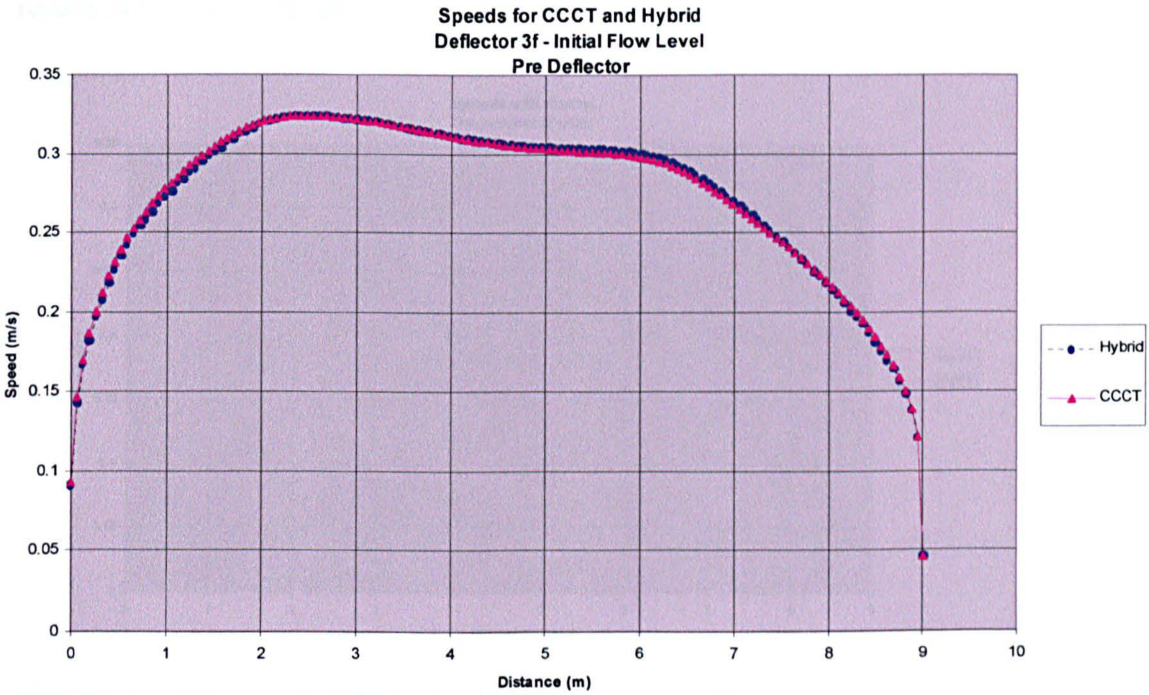


Figure 4.5 - Comparison of speeds produced using Hybrid and CCCT differencing at section 4

To solve a flow problem using CFX, the flow domain is subdivided into a number of cells. In the case of the three deflector sites that were modelled in this thesis, the vertical plane (depth) was broken down into 30 subdivisions, the channel width into approximately 60 subdivisions and the length of each of the reaches into approximately 120 subdivisions. This produced approximately 2 x 10^5 flow cells. Trials with different numbers of subdivisions in different directions revealed this to be the best layout for obtaining a converged solution. In addition, it was considered necessary to use as many cells in the vertical plane as possible so as to obtain the best representation of the vertical velocity profile in the bottom 20% of the flow depth for calculation of the bed shear stress. This matter is explained further in chapter 6.

In order to check for accuracy and grid independence the total number of grid cells was doubled in a test case. The effect on the solution was found to be negligible. The results are shown in figures 4.6 and 4.7.

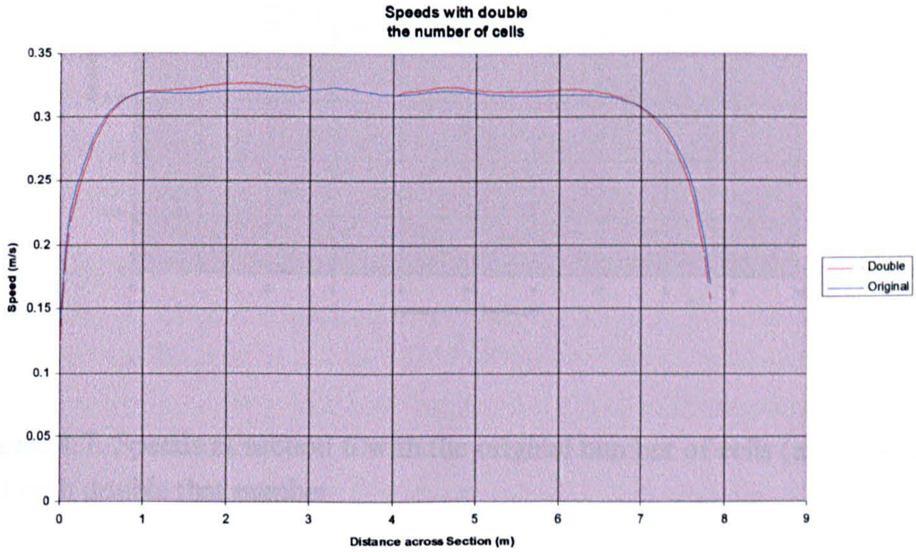


Figure 4.6. Speeds at section 5 with the original number of cells (approximately $2x10^5$) and with double that number

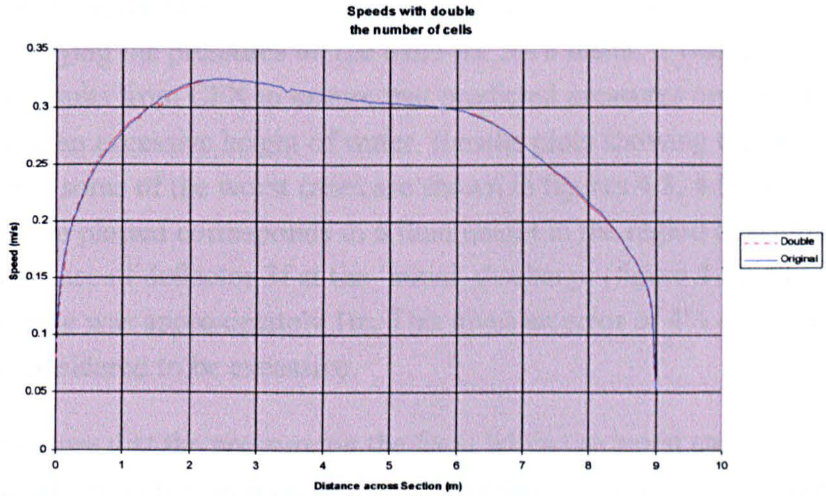


Figure 4.7. Speeds at section 6 with the original number of cells (approximately 2×10^5) and with double that number

Each simulation was run for 2000 iterations. To ensure convergence had occurred the flow residuals (the three momentum components, mass source, turbulence energy and energy dissipation) were checked at the end of each simulation. These were as low as 1×10^{-3}, and had always reduced by a factor of 1×10^5 between the second and last iterations.

CFX is capable of calculating the position of the free water surface itself, as part of the simulation run, given all the other relevant information (boundary conditions etc). However, it was found that this was not the case for this thesis, and in fact the position of the free surface was 'blurred'. Moving vertically upwards, the model predicted a transition region at the level where the water surface was expected. In this region, the model predicted a very gradual change (over a height of a few tens of centimetres) from 100% water and 0% air, up to 100% air 0% water. Clearly, this is not the case in reality. To overcome this, the position of the free surface predicted for each case by the one dimensional model ISIS, was used as a fixed boundary on top of the flow in the corresponding case in CFX. This is a reasonable procedure to adopt since CFX is capable of calculating a varying pressure on this fixed lid (for example to represent the superelevation of the water surface that would be expected on the outside of a bend). The pressure that it predicts on the lid corresponds to a height of water (pressure head). Thus, the effect of CFX predicting a different pressure on the lid appears to be the same as if it were to actually alter the position of the free water surface.

There would be no difference between altering pressures on the fixed lid and actually moving the position of the free water surface, if it were not for the mass continuity equation. Considering this equation, fluid would be needed to occupy the volume created by a rise in the position of the free water surface. This fluid is not taken into account by varying the pressures on the fixed lid. As a result, it was necessary to monitor the results from CFX to ensure that predicted pressures on the lid did not correspond to an excessive height of water. Results plots showing the pressures near the fixed lid in some of the worst cases are shown in figures 4.8, 4.9 and 4.10. The largest pressure plotted corresponds to a fluid height in the region of 0.04m or 4cm, this is for the case of deflector 3f at the 'initial' discharge (figure 4.8). The depth of the river in this case was approximately 1m. This gives an error of 4% over the flow depth. This is not considered to be excessive.

In order to ensure that the pressure on the fixed lid, in the worst case, was not having an excessive effect on the solution, a test was carried out. This involved raising the elevation of the fixed lid at points where the largest excess pressures are predicted. These occur in the vicinity of the deflector. The effect of raising the elevation of the water surface, at a number of localised points, on the plotted lid pressures is shown in figure 4.11. The pressures have become equalised over the region plotted. The effect on flow speeds is shown in figures 4.12 and 4.13, which are plotted at the water surface. Figure 4.12 shows the flow speeds prior to altering the elevation of the fixed lid, whilst figure 4.13 shows the speeds after the alteration. In figure 4.13, the flow speeds have reduced due to the fact that the cross sectional area of flow has increased. The difference in speeds is in the order of 4%, which is the same error as that in the depth. This is not considered to be excessive, and in all other cases the position of the fixed lid was not altered from that obtained from the one dimensional model.

The post processor VIEW only allows results to be plotted on a horizontal plane. In order to plot the pressures on the lid throughout the solution, a sloping surface would be required. Thus, the plotted pressures are those at the fixed lid at the downstream boundary. Moving towards the upstream boundary, the horizontal plane on which the results are plotted falls a little below the position of the fixed lid. This scenario is shown in figure 4.14.

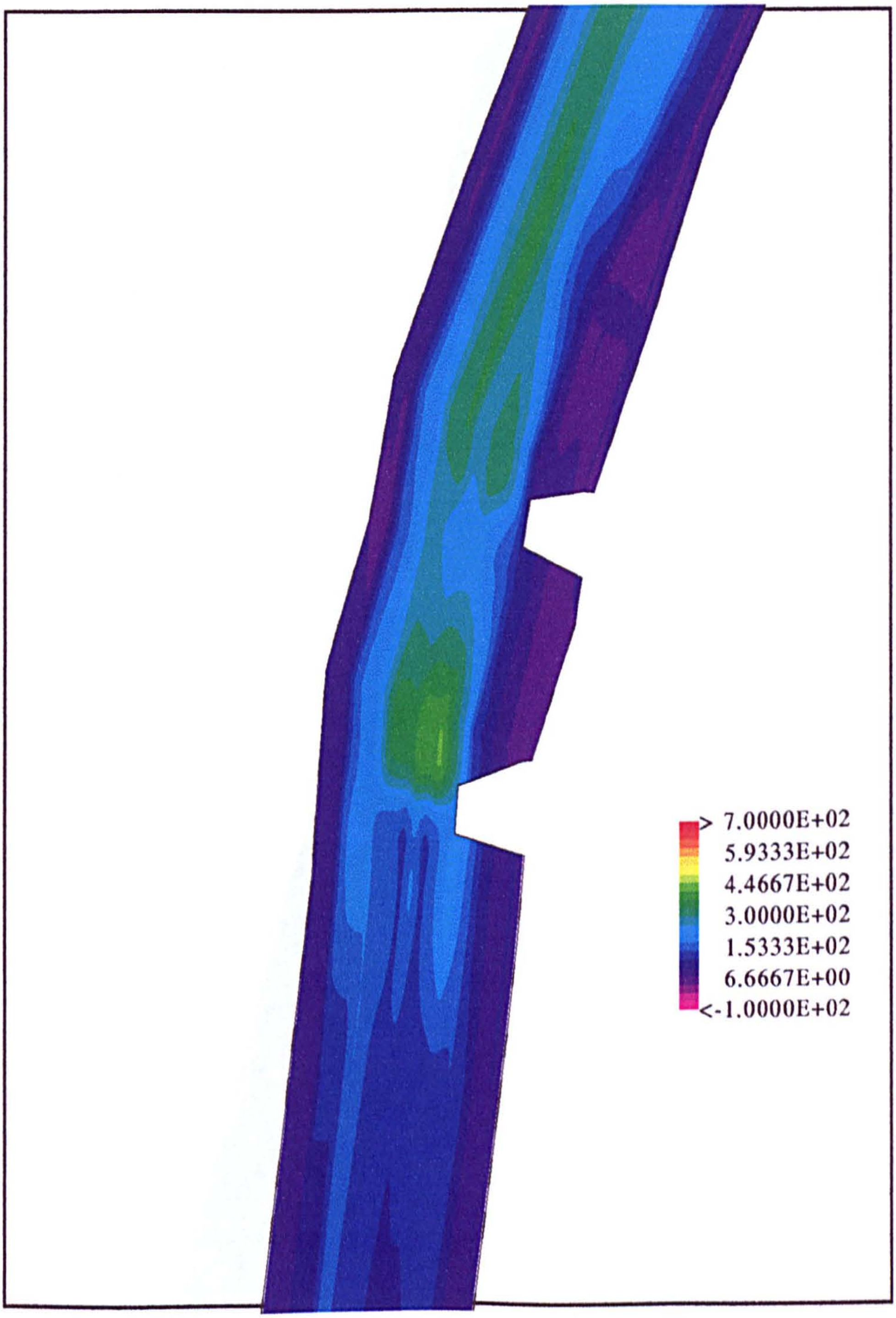


Figure 4.8: Pressures near the fixed lid for the 'Initial' Discharge - Deflector 3f

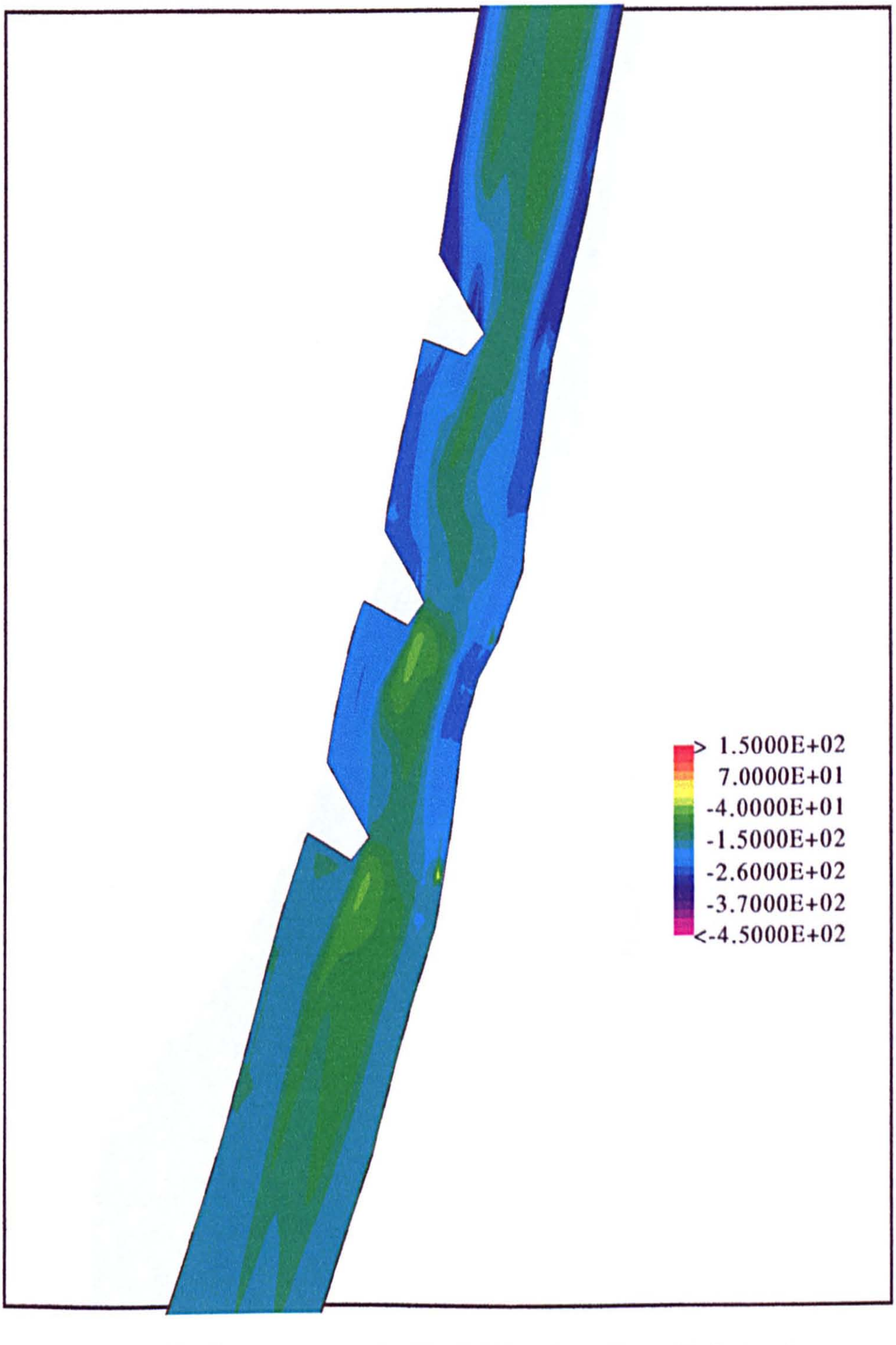


Figure 4.9: Pressures near the Fixed Lid at Low Flow - Deflector 3c

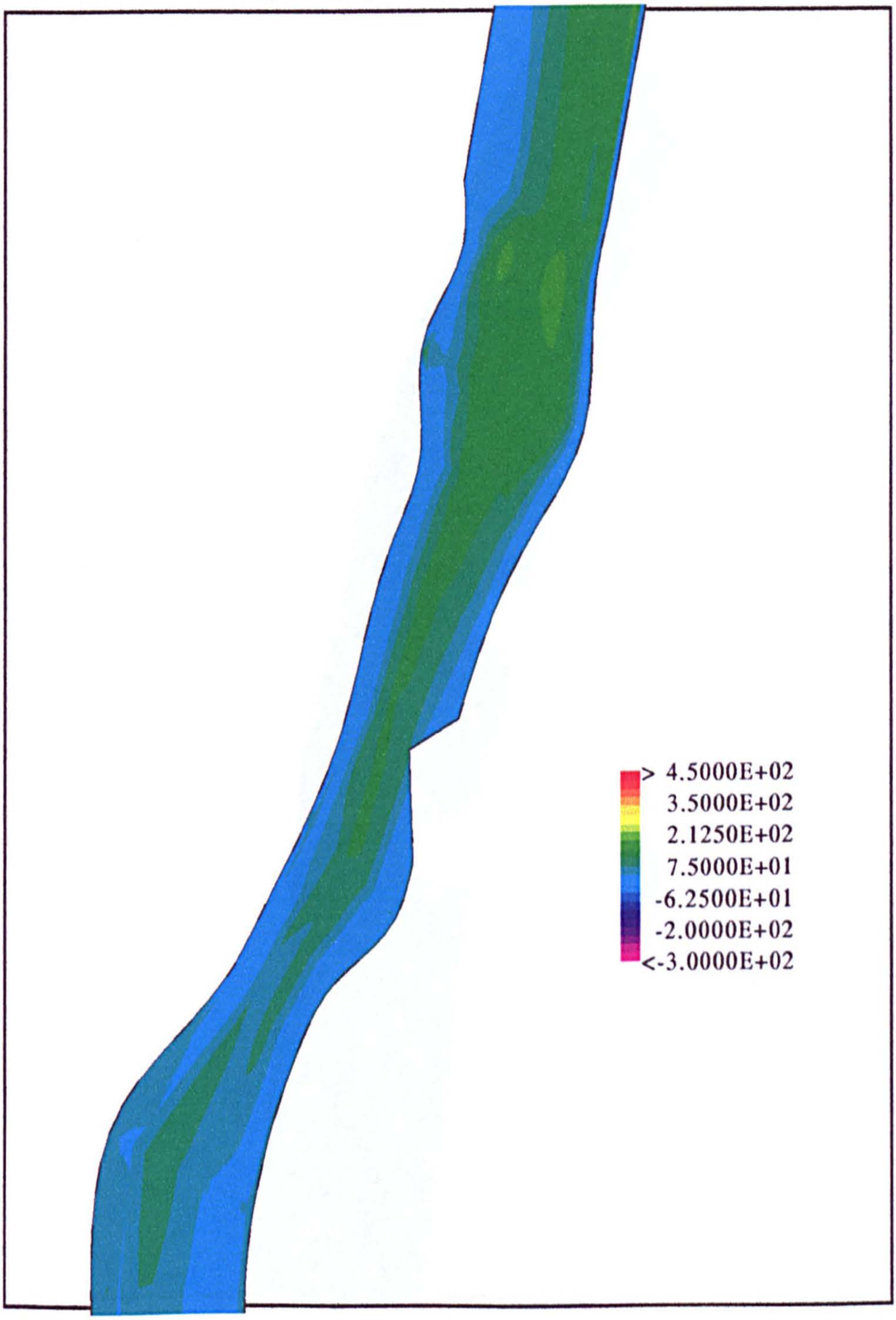


Figure 4.10: Pressures near the Fixed Lid at Extreme Low Flow - Deflector 6a

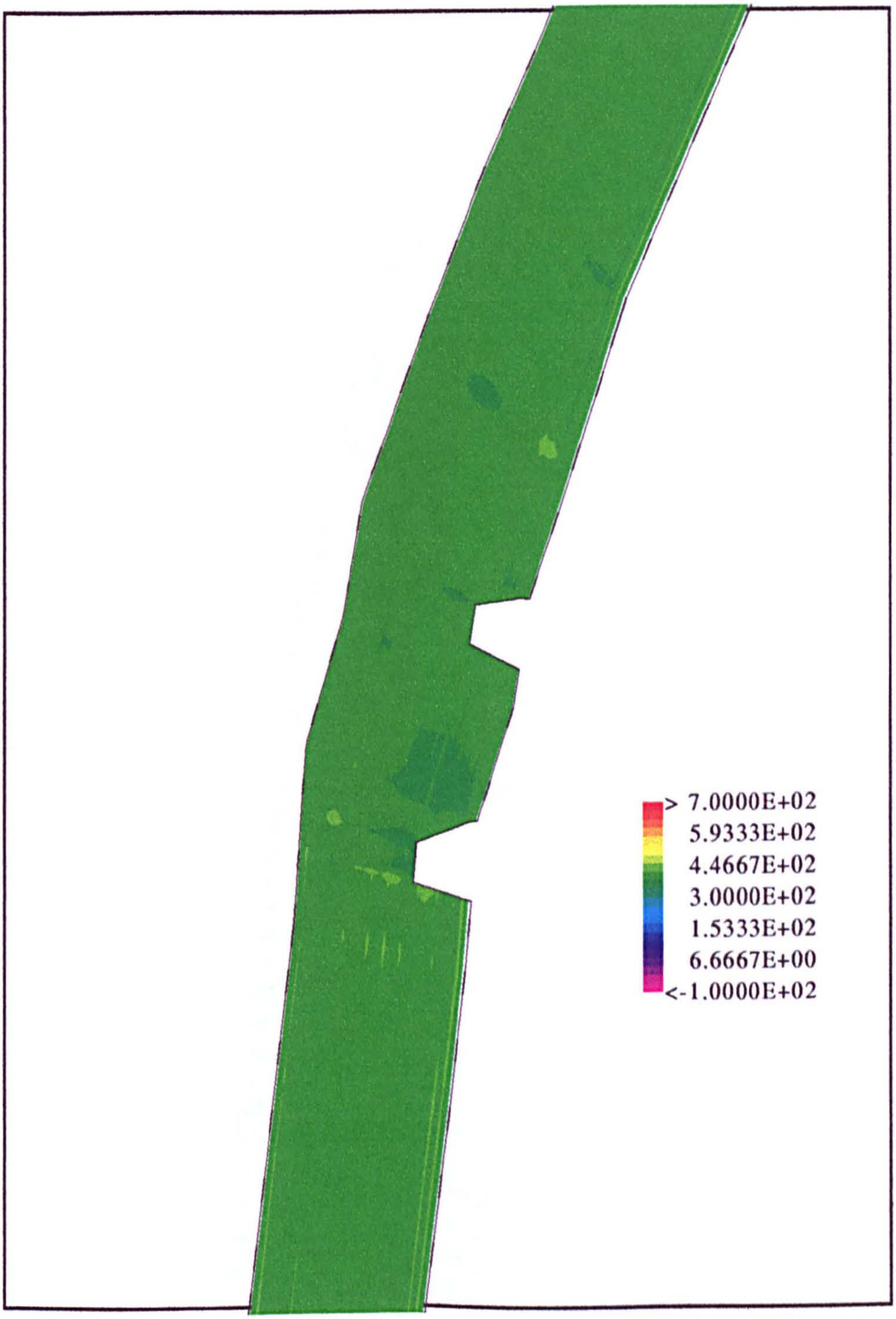


Figure 4.11: Pressures near the fixed lid for the 'Initial' Discharge with the elevation of the water surface raised at the deflectors to eliminate the pressure differential - Deflector 3f

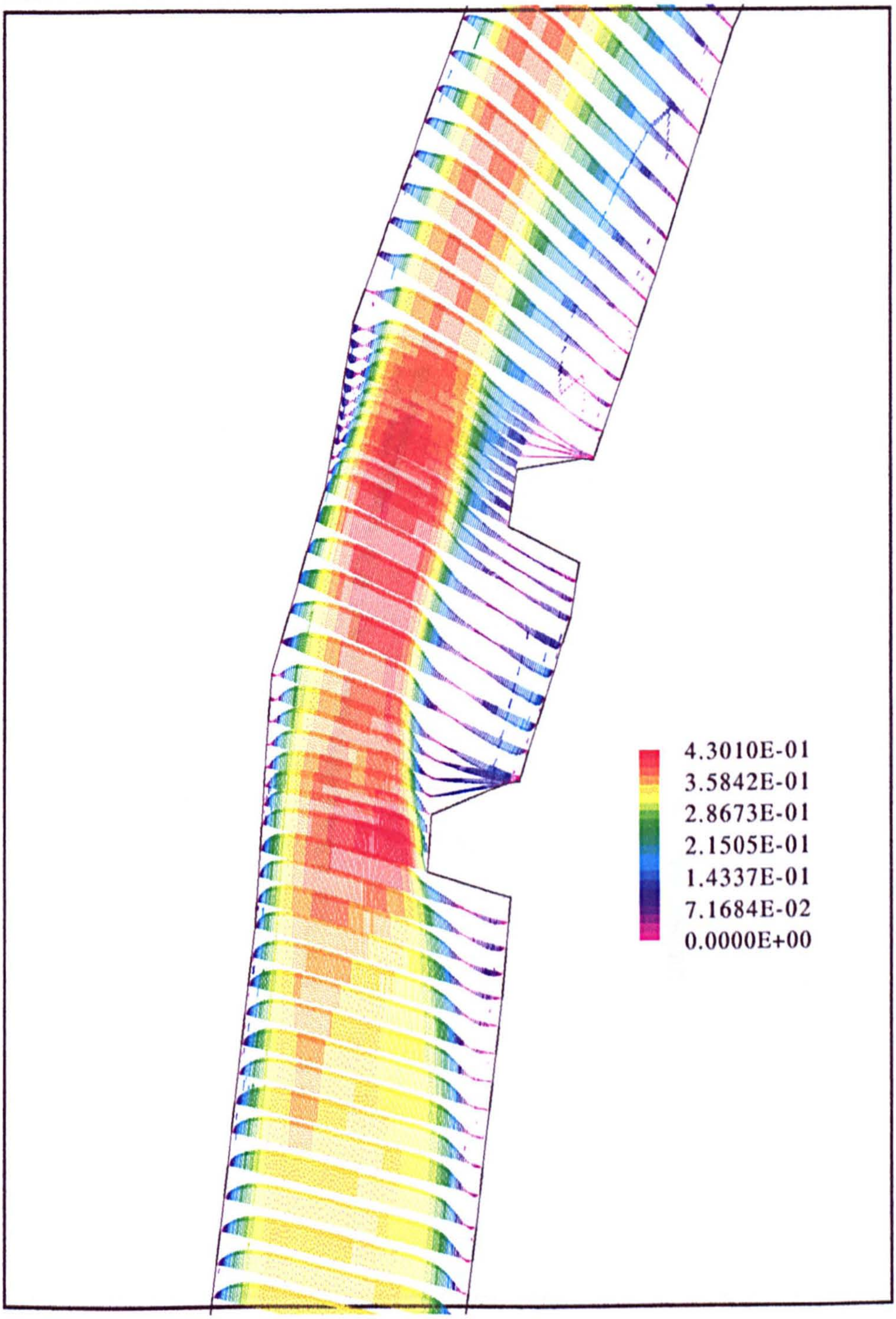


Figure 4.12: Speeds near the fixed lid for the 'Initial' Discharge - Deflector 3f

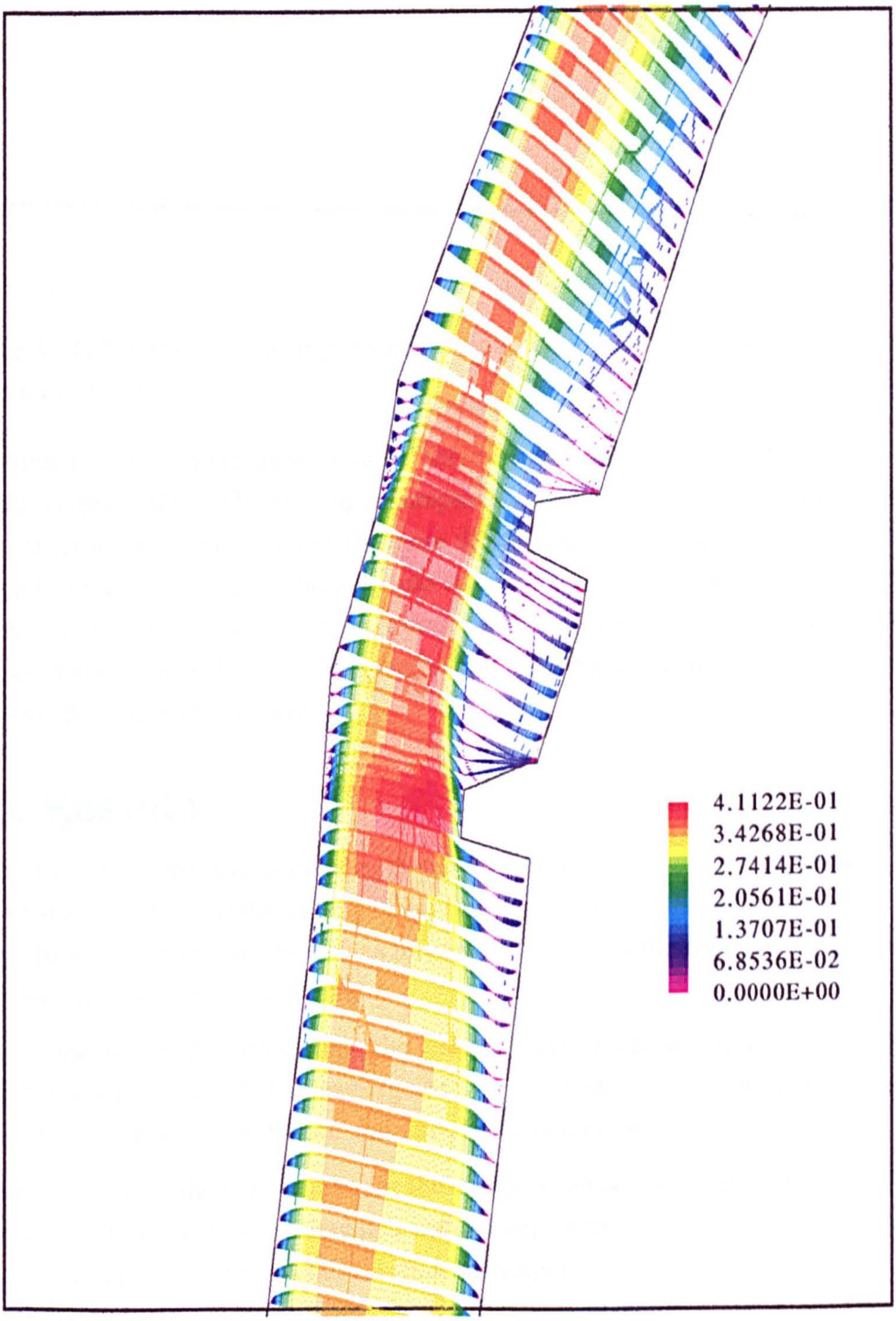


Figure 4.13: Speeds near the fixed lid for the 'Initial' Discharge with the water surface elevation altered at the deflectors to eliminate any pressure differential - Deflector 3f

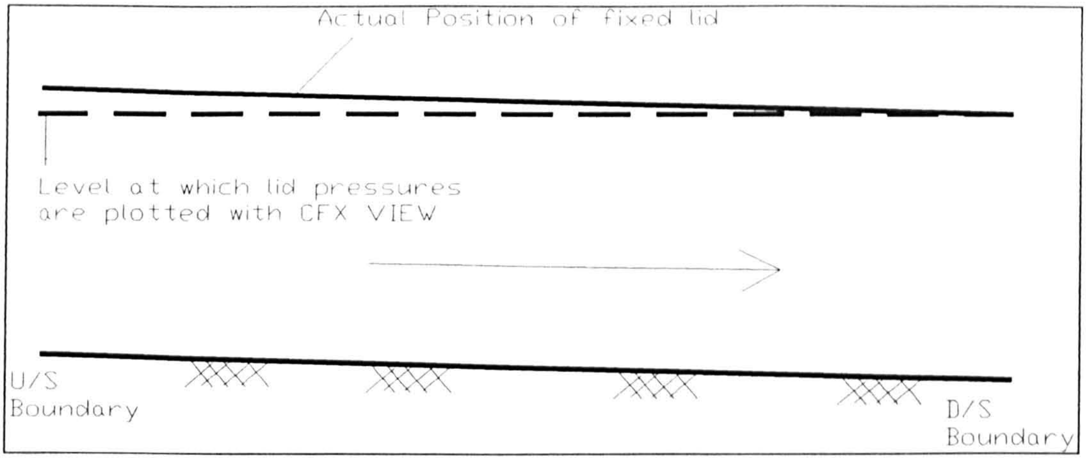


Figure 4.14. Location at which pressures are plotted in figures 4.8, 4.9, 4.10 and 4.11 using CFX VIEW

The three programs were used to model three deflector sites. Each site is modelled as an individual problem. This is a big assumption as it dispenses with backwater effects from structures downstream. For the three dimensional modelling, it also takes no account of the flow structure coming into the modelled reach from upstream, or the effect on the flow structure of the downstream channel geometry. It was necessary to treat each site separately or the problem would have been too large to simulate using the three dimensional software.

4.5. Results

Table 4.1 summarises the discharges that were simulated at the three individual deflector sites. These problems constitute the two and three dimensional modelling cases studied in this thesis, as well as forming the basis for the habitat and sediment modelling outlined in chapter 5 and 6.

For the one dimensional modelling, the whole of section six (incorporating five deflectors) was modelled at a discharge of 8 m^3/s. All the cases mentioned were simulated for the situation before and after the deflectors were installed.

Figure 4.15 shows the ISIS results for the whole of section six in the form of a long section. The plot shows the calculated water levels in the section. Figure 4.16 shows a plot of the same set of results but this time as output from HEC-RAS.

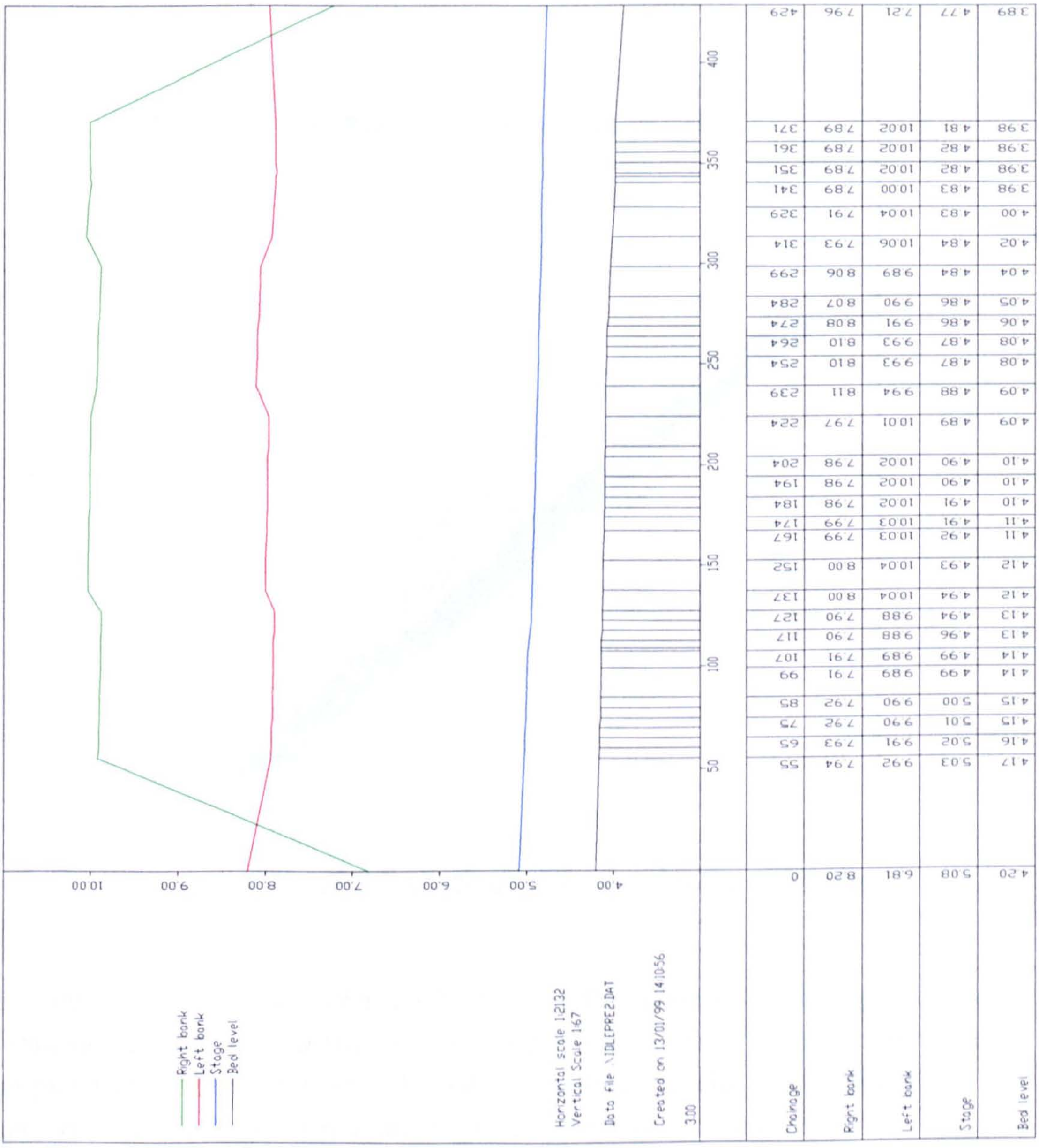


Figure 4.15. ISIS Results for the whole of section 6 pre deflectors.

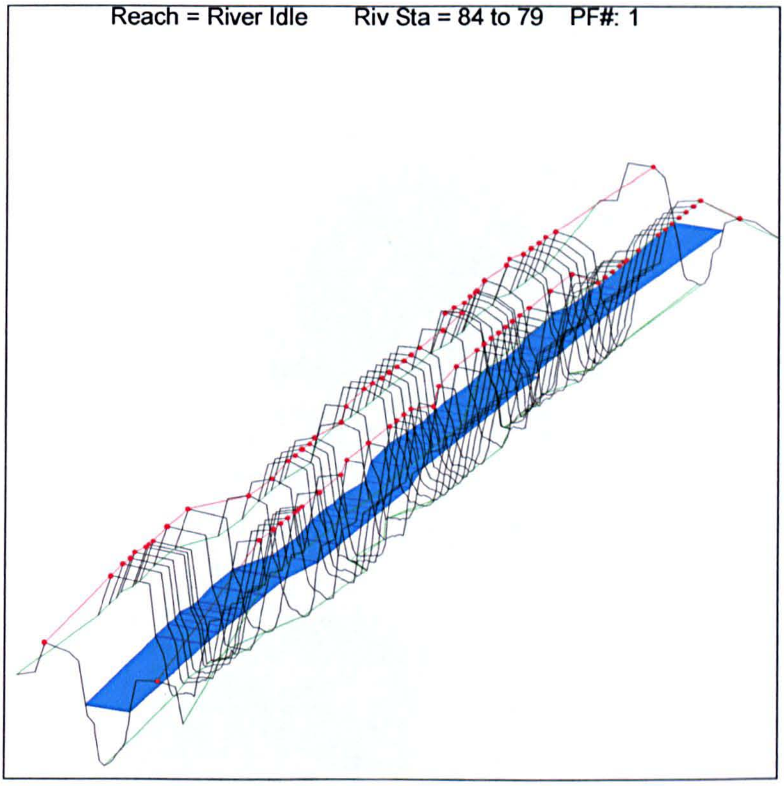


Figure 4.16. HEC-RAS Results for Section 6 Pre Deflectors.

Results plots 4.17, 4.18, 4.19 and 4.20 show the flow patterns for the depth averaged velocities, at each of the deflectors, predicted by SSIIM. Plot 4.20 is the same location as plot 4.18 (deflector 3c) but with double the number of cells. The predicted flow pattern is identical. This demonstrates grid independence in the solution.

It should be noted that the orientation of the plots is different to CFX. This is because SSIIM requires the format of the data to contain increasing chainage between cross sections, from left to right across the plots in each case.

The representation of the geometry of the deflectors is a slight approximation with SSIIM as the program requires there to be a constant number of cells across the channel width, at all cross sections. Thus it was necessary to taper in the channel bank at the upstream edge of each deflector for sites 3c and 3f (plots 4.18 and 4.19). This is considered to be an acceptable approximation since the artificial narrowing of the channel at the upstream end of the deflector follows a natural streamline within the flow.

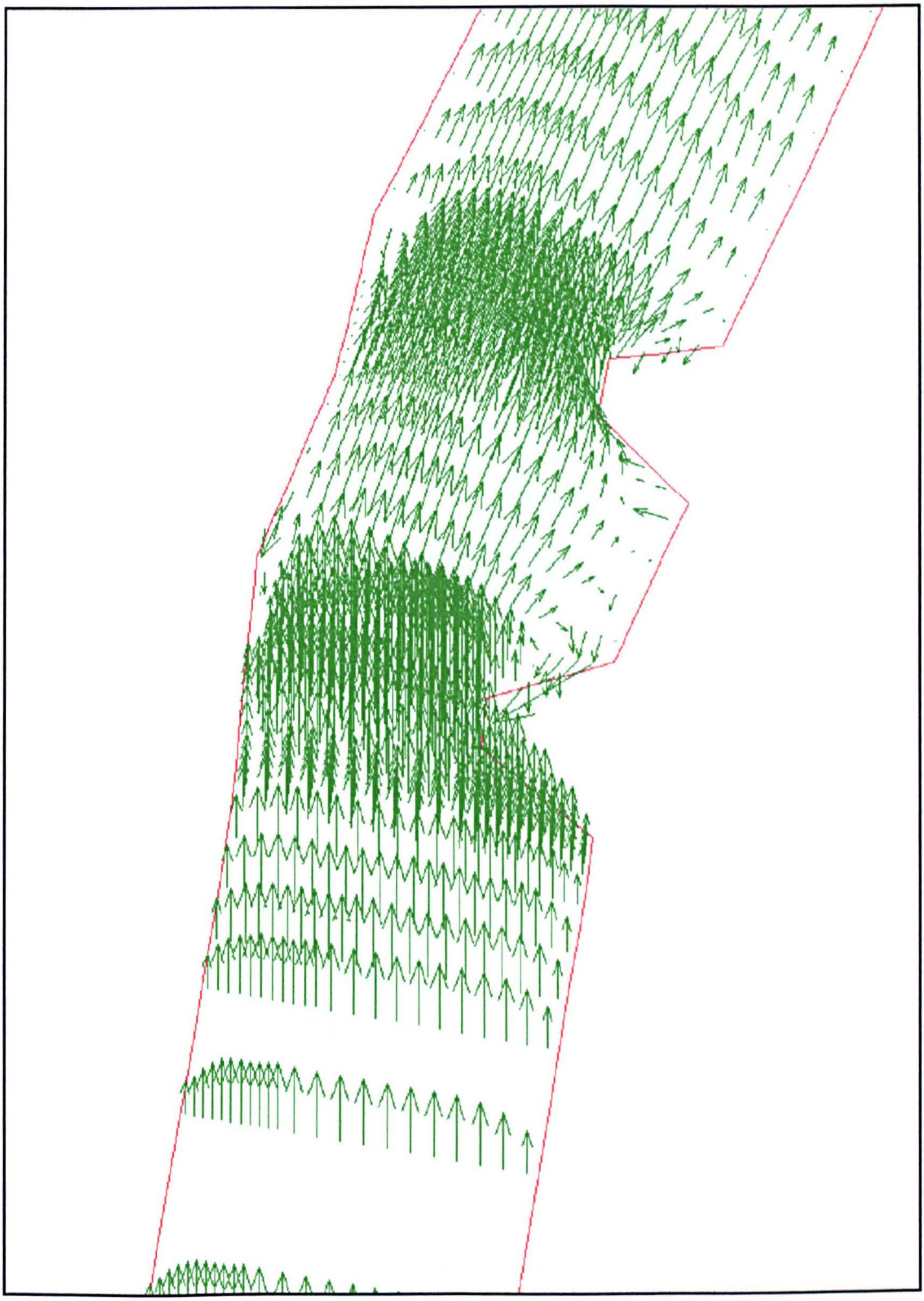


Figure 4.17. Velocity pattern predicted by SSIIM at deflector 3f

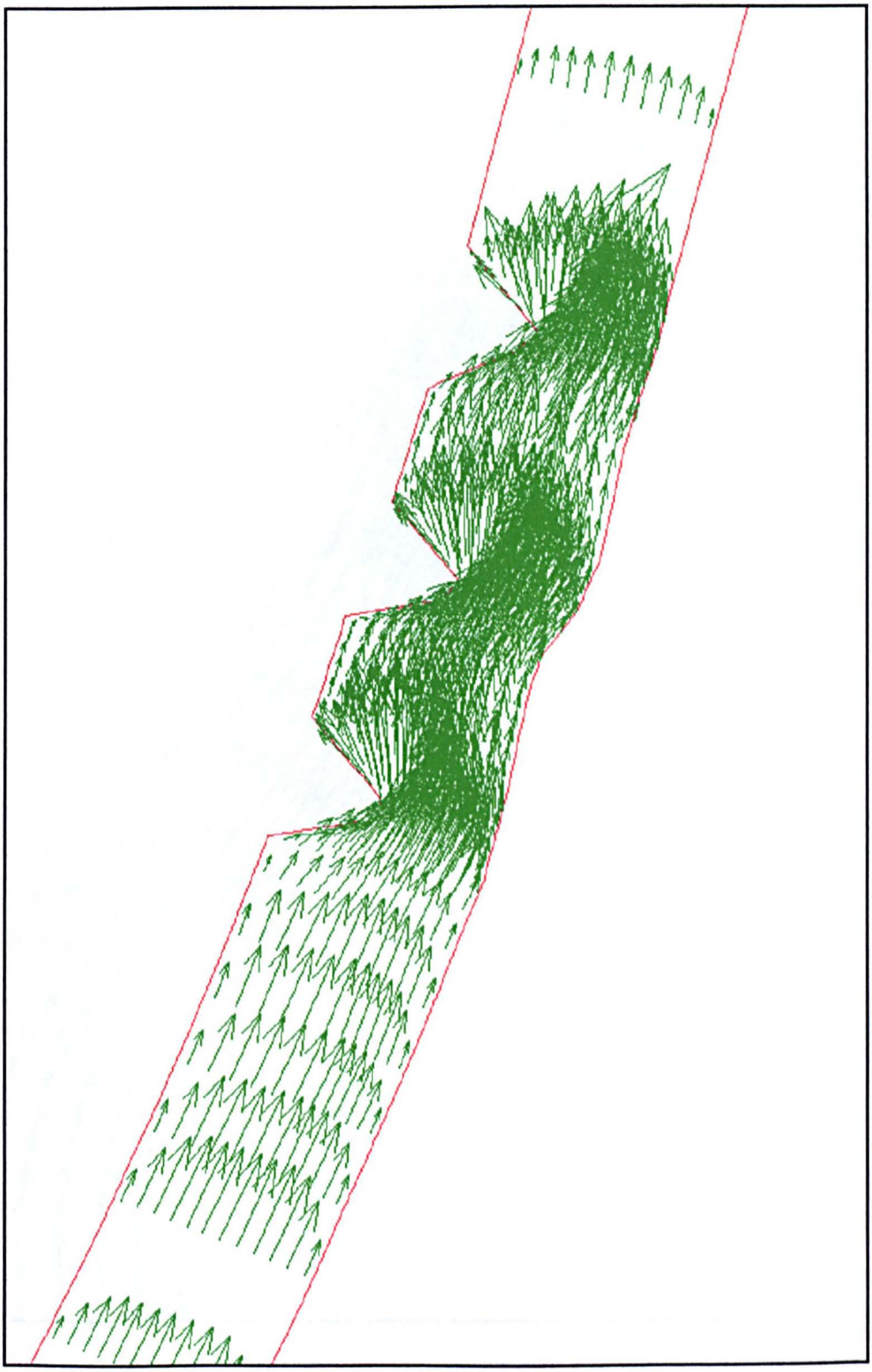


Figure 4.18 Velocity pattern predicted by SSIIM at deflector 3c

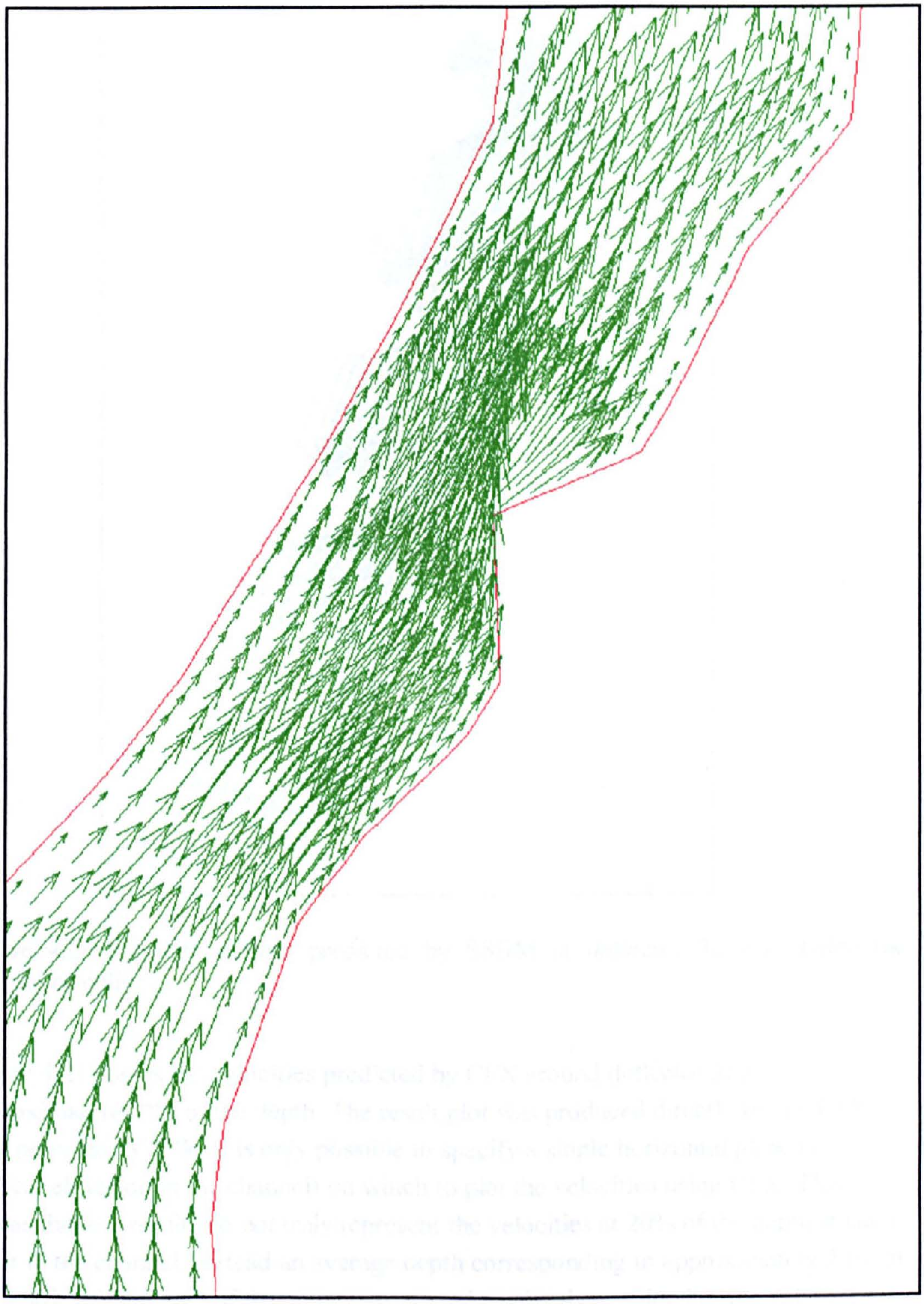


Figure 4.19 Velocity pattern predicted by SSIIM at deflector 6a

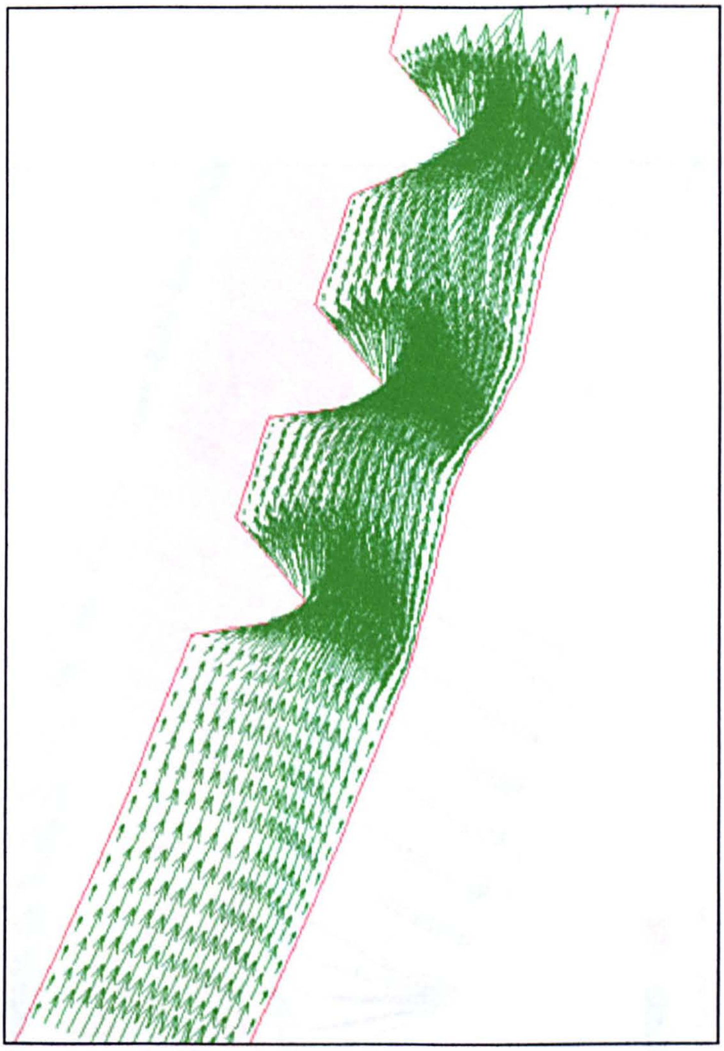


Figure 4.20 Velocity pattern predicted by SSIIM at deflector 3c with twice the number of cells.

Figure 4.21 shows the velocities predicted by CFX around deflector 3f at approximately 20% of the depth. The result plot was produced directly by the CFX post processor VIEW. It is only possible to specify a single horizontal plane (a vertical elevation in the channel) on which to plot the velocities using CFX. This means that the results do not truly represent the velocities at 20% of the depth at each point in the channel. Instead an average depth corresponding to approximately 20% of the whole of the depth of the channel was used to plot the results. For the validation later in this chapter, a FORTRAN program was written to extract the velocities at 20% depth etc at each point in the channel from the CFX output. This is discussed later in this chapter in the section on validation. Figure 4.23 shows the CFX predicted velocities for 20% depth at deflector 3c, and Figure 4.24 shows the corresponding plot for deflector 6a. Figure 4.22 shows the same information as is 4.21, but in this case the simulation has been run without any turbulence model (i.e. laminar flow).

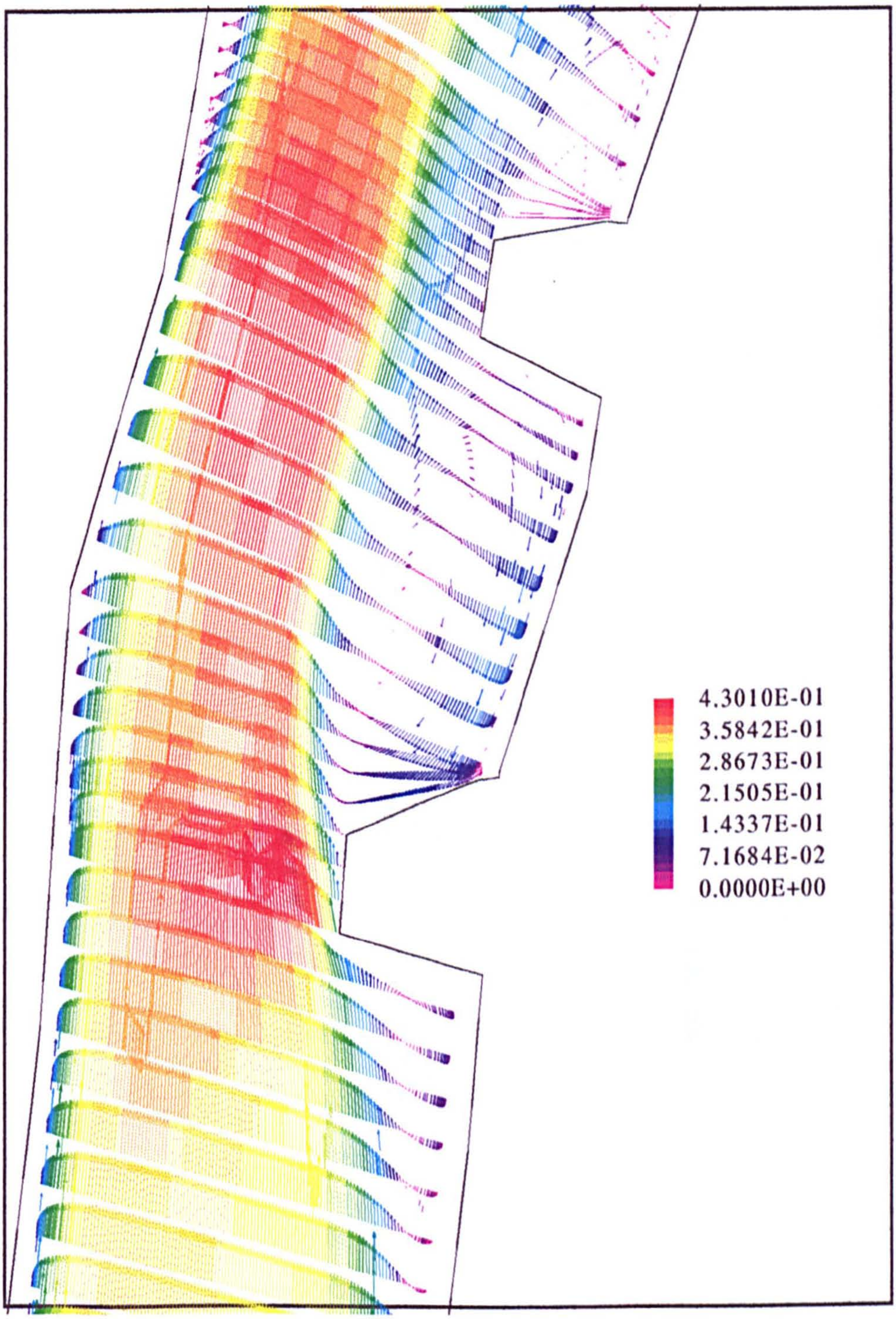


Figure 4.21: Velocity pattern predicted at 20% depth by CFX - deflector 3f

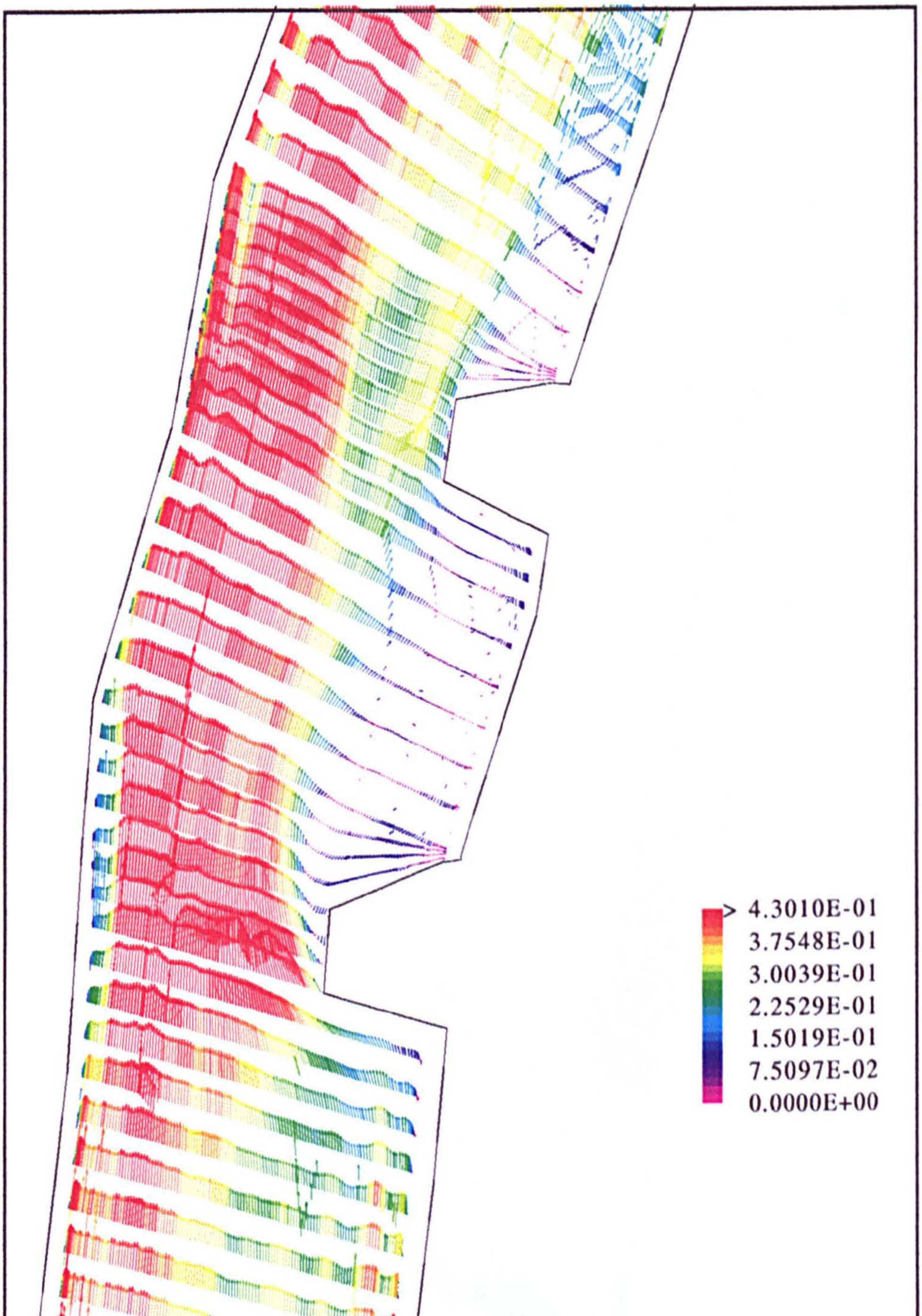


Figure 4.22: Velocity pattern predicted at 20% depth by CFX with laminar flow - deflector 3f

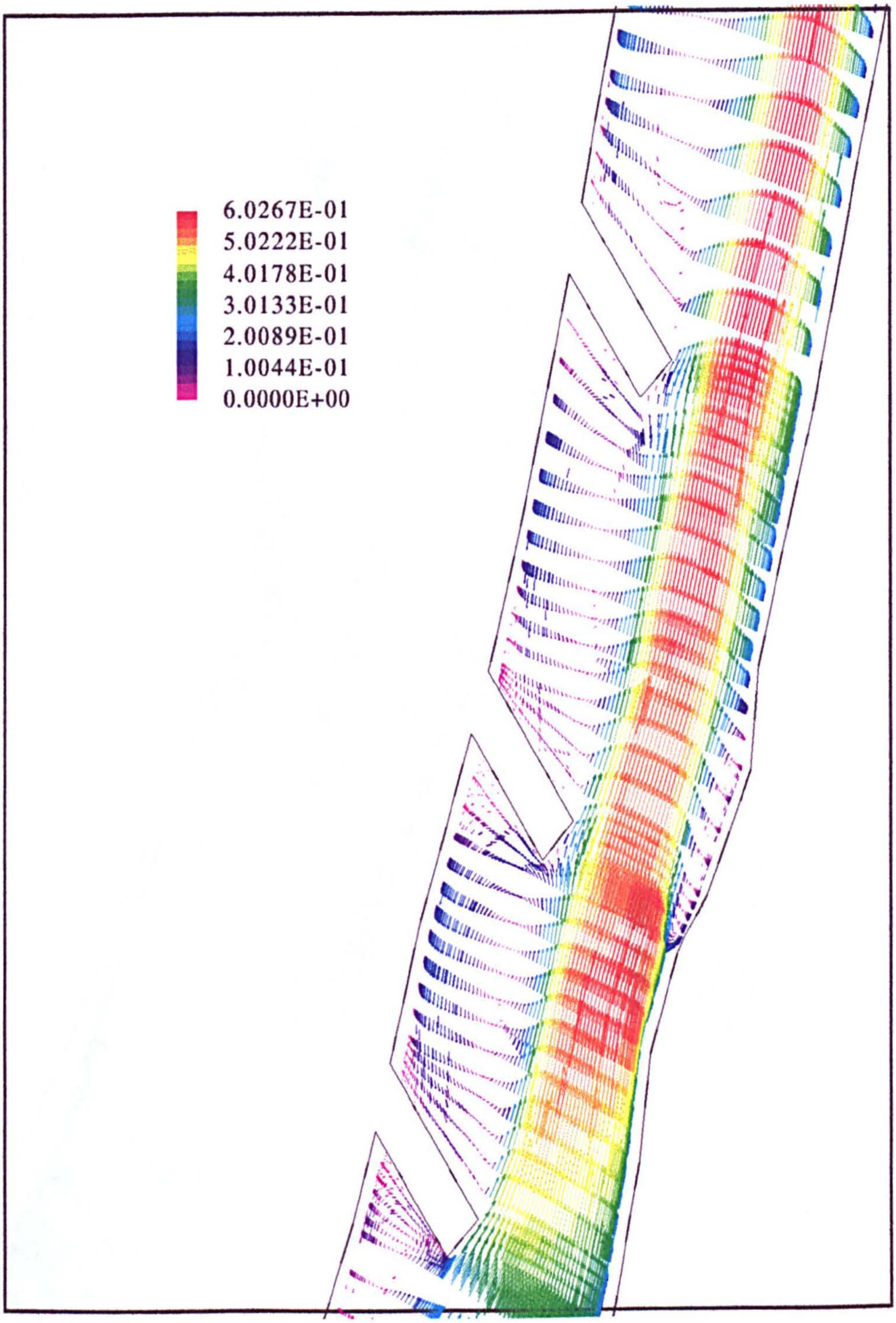


Figure 4.23: Velocity pattern predicted at 20% depth by CFX - deflector 3c

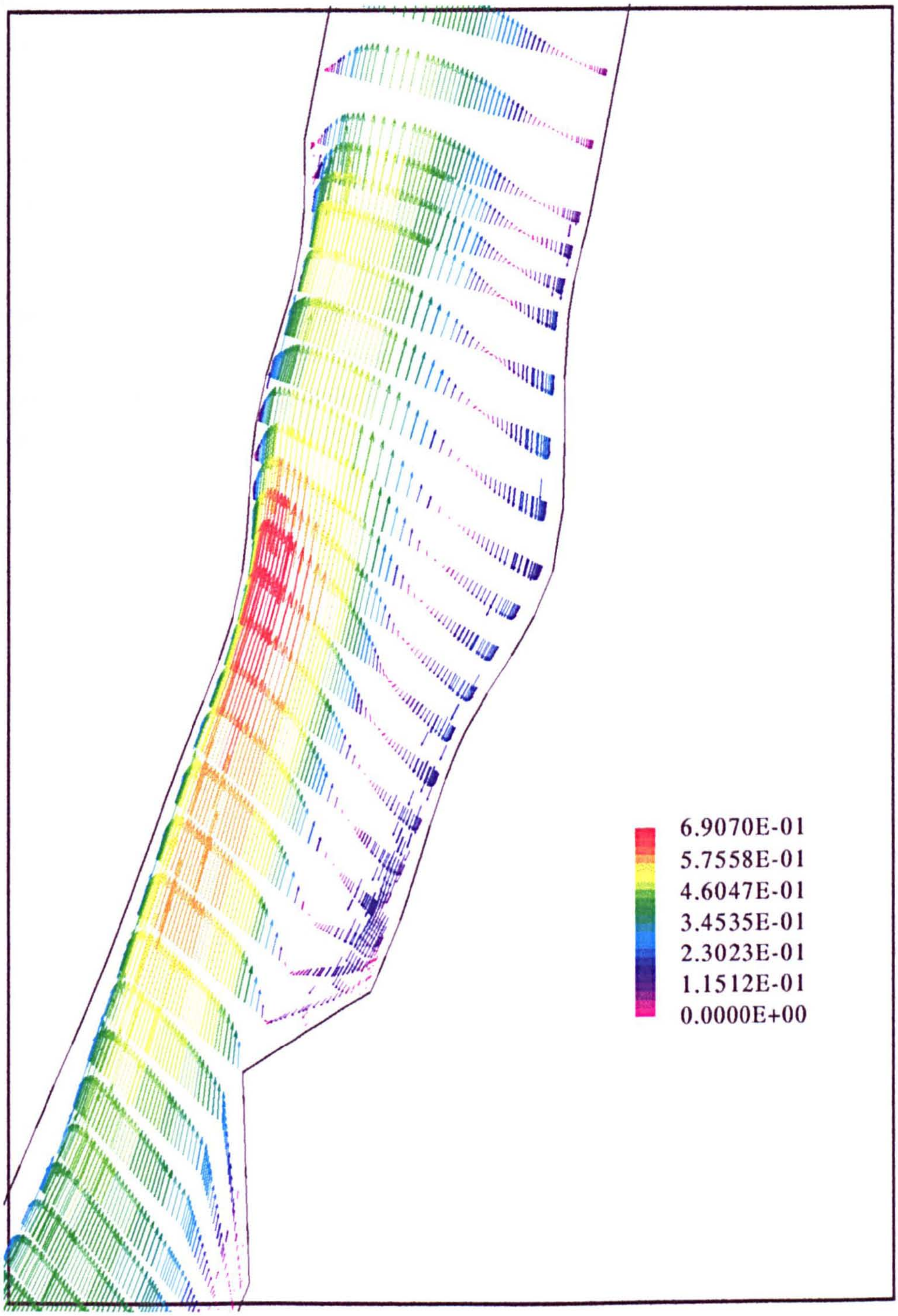


Figure 4.24: Velocity Pattern predicted at 20% depth by CFX - Deflector 6a

4.6. Validation of Results

No validation of the one dimensional results produced is possible. It would be possible to compare the panel velocities predicted by ISIS with site measured velocities but the one dimensional velocities would inevitably be incorrect where any secondary flow occurs e.g. at a bend or in the vicinity of a deflector. This has been well documented in earlier research work (Acreman, 1998) and is not repeated here. However, a comparison of the results from ISIS and HEC-RAS for the increased water surface elevation for the whole of section six following the installation of the deflectors is included.

Validation of two and three dimensional results is carried out by comparing predicted velocities against site measured velocities which were recorded at the 'initial' discharge. The site measurements of velocity were taken by students of the School of Geography at the University of Nottingham (Bromley, 1997). Results of the site measurements of velocity taken with a two dimensional flow meter and at 20% depth for deflector 3f are shown in Figure 4.25.

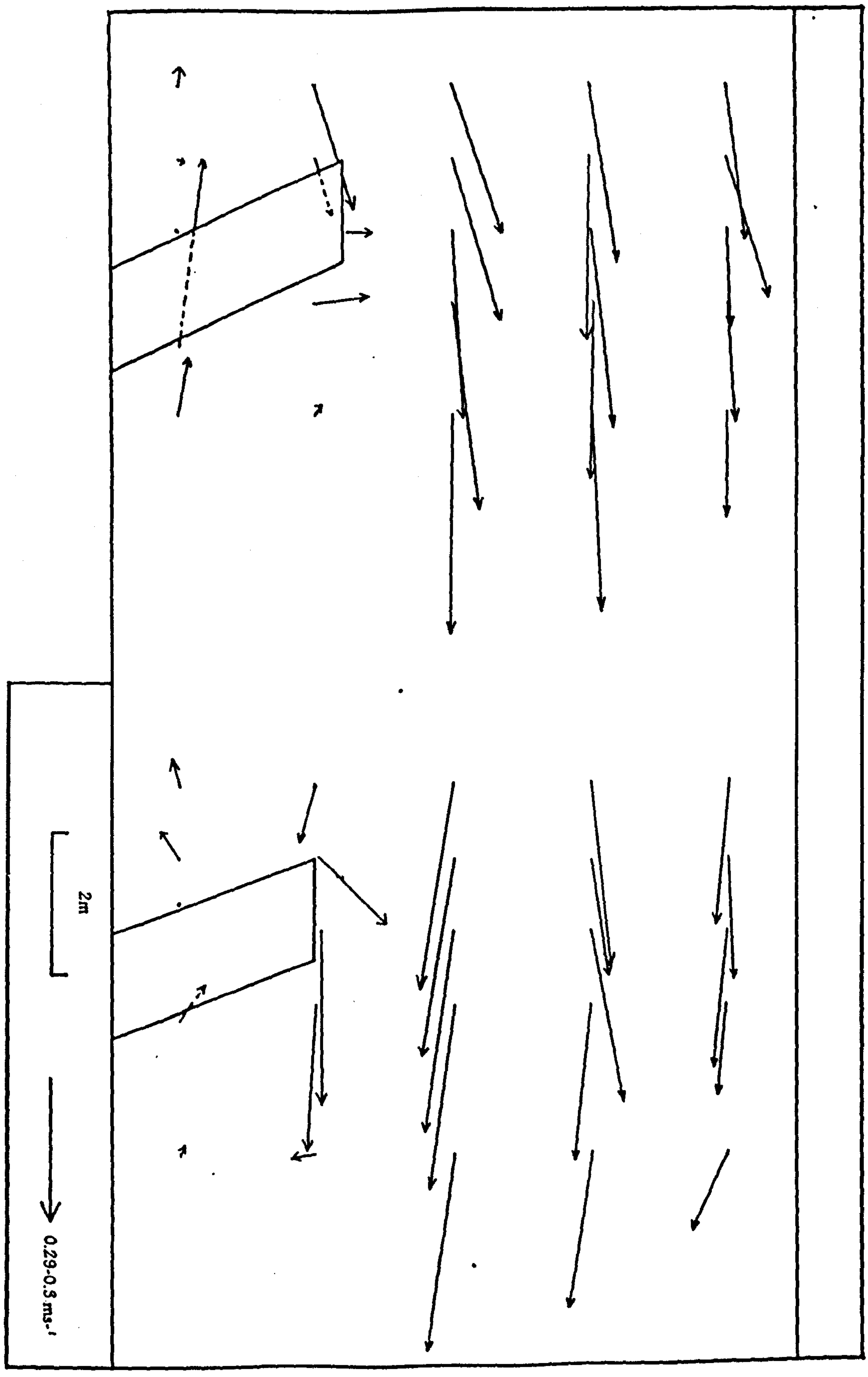


Figure 4.25 Site measured velocities at 20% depth for deflector 3f.

4.6.1 One Dimensional

As has been stated, there appears little point in comparing one dimensional velocities with site measured velocities since it is generally accepted that they will not be correct in many cases, and this has been ably demonstrated in other research work. Comparing the predicted water elevations for the two one dimensional pieces of software (at a constant discharge of $8m^3/s$) in section six reveals a maximum discrepancy of only 0.01m in predicted water levels.

4.6.2 Two Dimensional

SSIIM predicts a depth averaged velocity for each cell in the solution domain. The difference between the magnitude (but not the direction) of these velocities and the site measured velocities is plotted in Figure 4.26 to 4.28. A positive value indicates the SSIIM predicted velocity is greater than the site measured velocity. The results are summarised in table 4.2 later in the chapter.

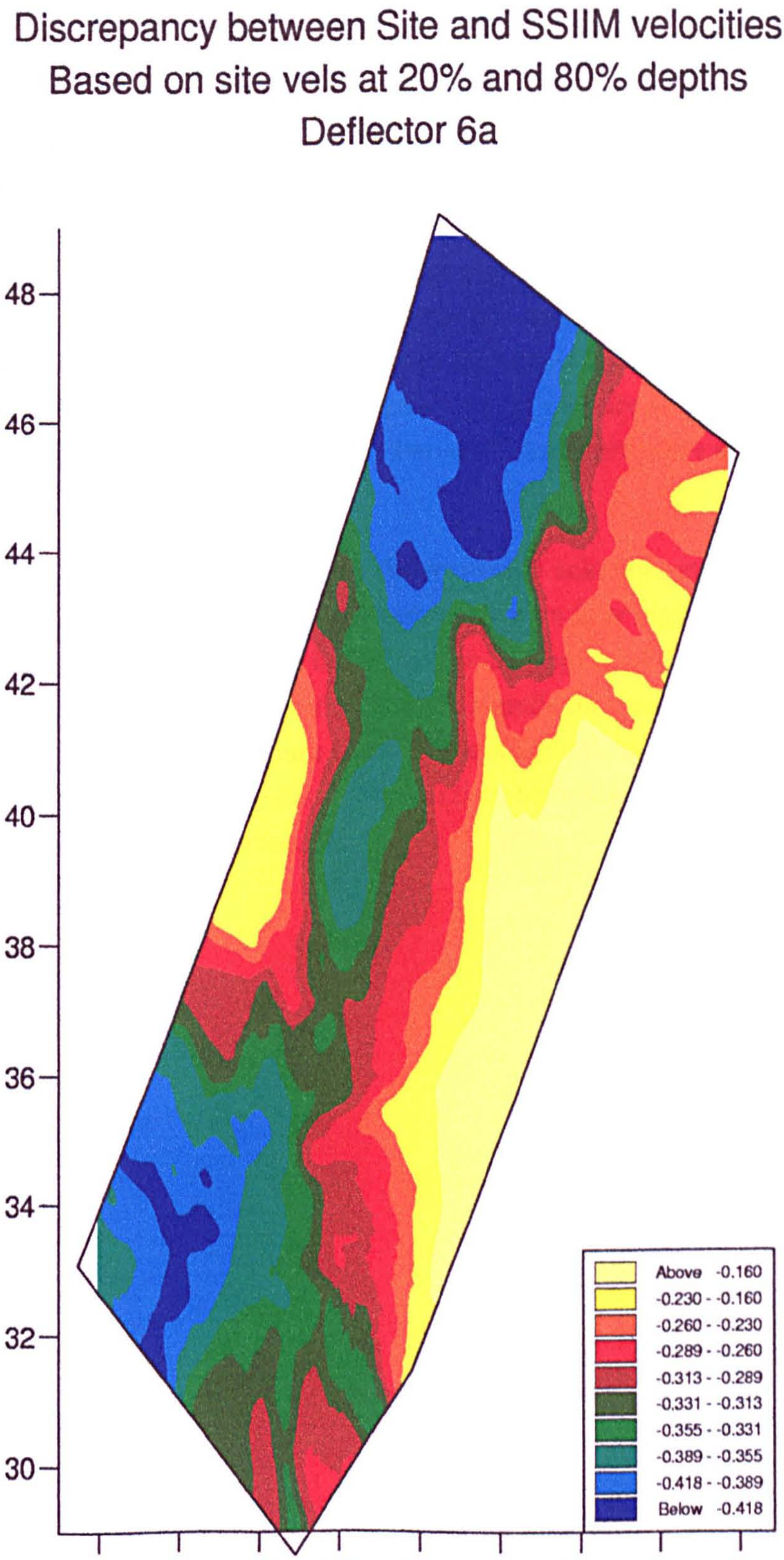


Figure 4.26: Validation of SSIIM velocities against site measurements for deflector site 6a

Site velocities were measured at 20%, 40% and 80% depth at deflector 3f, and at 20%, 60% and 80% depth for deflectors 6a and 3c (20% depth means the velocity at 20% down from the surface to the bed). British Standards suggest that the depth averaged velocity can be taken to correspond to the velocity at 60% depth, or the average of the velocities at 20% and 80% depth (ISO, 1979). Research by Smart (1999) showed that the velocity at 60% depth was a good indicator of the depth averaged velocity. Thus it is possible to use the site measured velocities in two ways, for deflector sites 3c and 6a, to compare with the values SSIIM predicts for the depth averaged velocity. Only one of each of these types of plot are included here, for each case, to save space. The results using the two different approaches are very similar in each case.

The procedure outlined above for using the measured velocities at 20%, 60% and 80% depths to generate depth averaged velocities assumes a fully logarithmic, hydrostatic pressure distribution. This will not be the case in the region of the flow deflectors where the flow structure is expected to contain significant three dimensional properties. However, the procedure is the best available for generating a depth averaged velocity, from the site data, that can then be compared with the two dimensional depth averaged velocities.

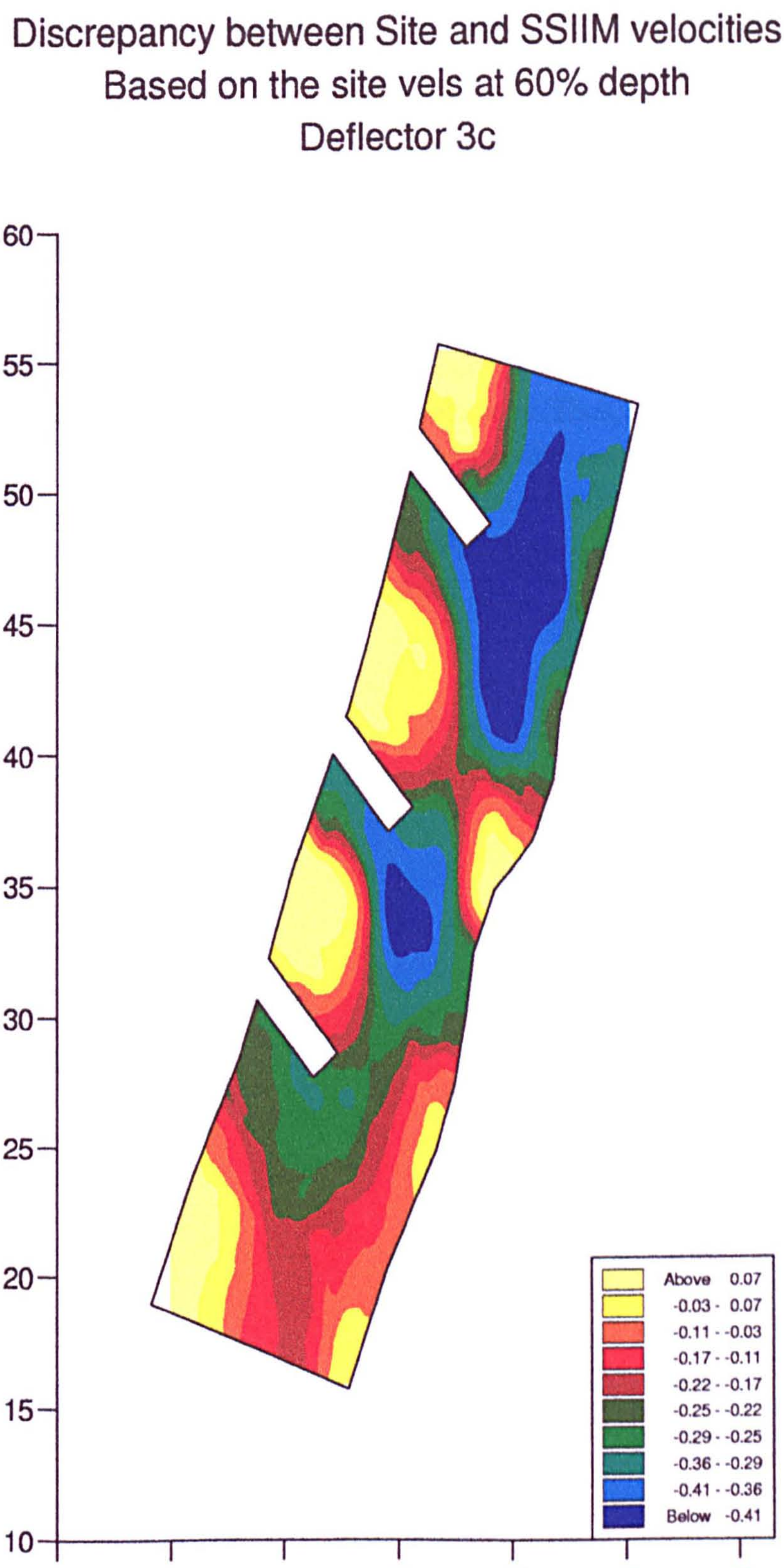


Figure 4.27: Validation of SSIIM velocities against site measurements for deflector site 3c

Calibration of predicted velocities against site measured velocities is only possible at the discharge present when the deflectors were surveyed (i.e. the 'initial' discharge). Also, since each deflector site was only modelled at one initial discharge (that being either the discharge present in the survey prior to the deflectors being installed or the first survey after installation) validation is only possible at the corresponding stage (either before or after deflector installation). Thus, comparison of SSIIM velocities with site velocities is provided at the 'initial' discharge and for site 6a before the deflectors were installed, and for both 3c and 3f after the deflectors were installed.

The site velocities were not measured at the same spatial plan co-ordinates as the SSIIM program generated its cell centered velocities. Thus, a small FORTRAN program was written by the author to calculate the four nearest SSIIM cell centroids to the location at which each site velocity had been recorded. The SSIIM velocity, at the location where the site velocity was recorded, was then calculated as the proximity weighted mean of these four SSIIM velocities.

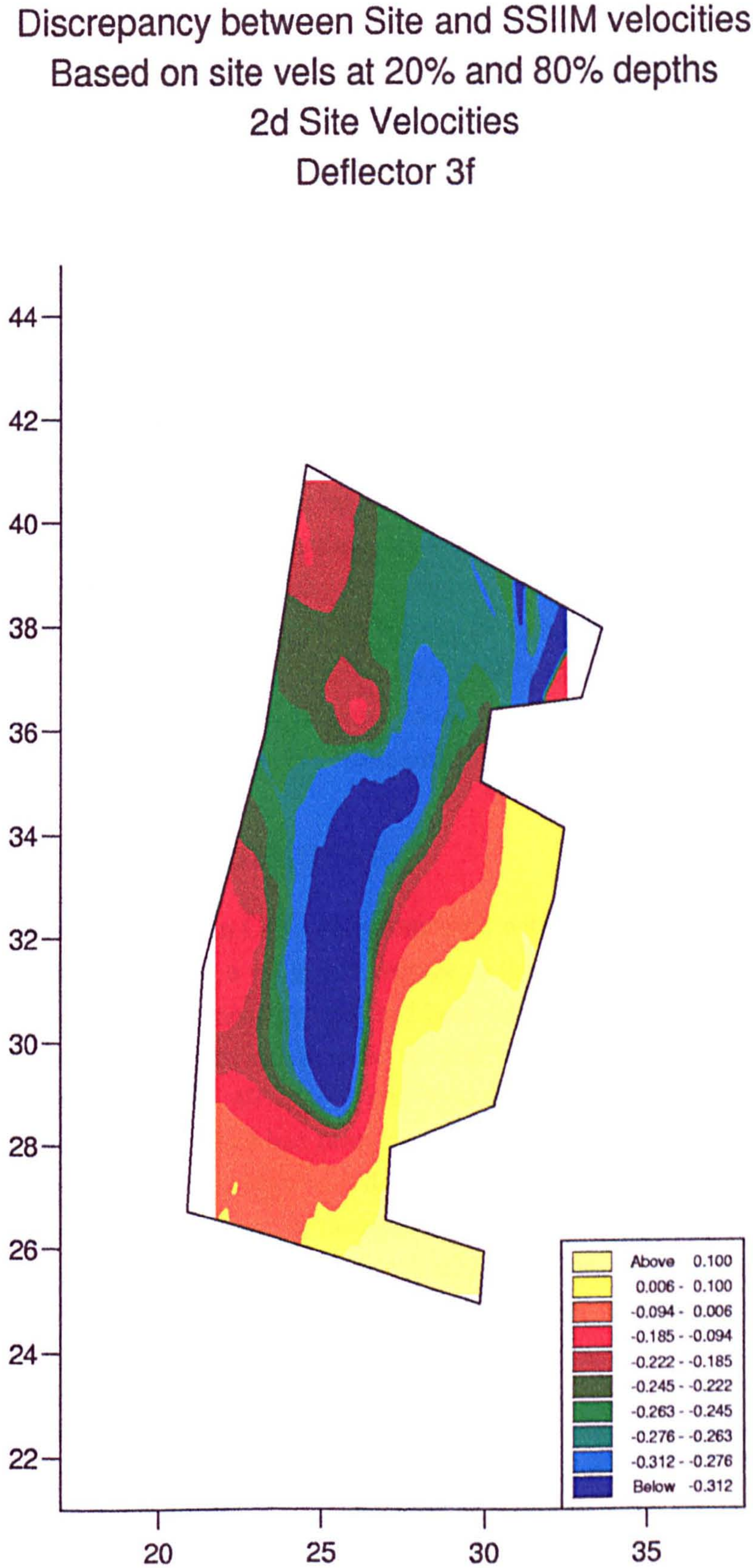


Figure 4.28: Validation of SSIIM velocities against site measurements for deflector site 3f

An important point to note concerning the site measured velocities is that the large majority of the readings were taken with a one-dimensional electromagnetic flow meter. This measures velocities only in the long-channel direction. The corresponding velocities predicted by both SSIIM and CFX are the resultant of two mutually perpendicular velocities. This would actually correspond to the resultant of the long-channel and cross-channel velocities. Thus, the validation process is not actually comparing like for like velocities in most cases. However, it is reasonable to expect that the long channel velocity will be considerably larger than the cross channel velocity in most locations and, thus, the validation should at least yield some useful comparison between the two sets of values. In any case, the one-dimensional velocity readings are the only ones that were taken in most cases and it is not considered to be practicable to reduce the SSIIM results to a set of long-stream and cross-stream velocities. A small number of velocity measurements were taken in both the long channel and cross channel directions at deflector 3f immediately in the vicinity of the deflectors. These consisted of five readings per cross section taken at a total of ten cross sections. This allows the derivation of a set of resultant velocities which can be compared with the SSIIM and CFX results on a like for like basis.

4.6.3 Three Dimensional

The predicted velocities from CFX were compared with the site measured values in a similar way to SSIIM, using a weighted average of the four nearest values to the location at which each site measurement had been taken. Resultant velocities were calculated at each cell centre (ignoring the vertical velocity component). In order to obtain the velocities at 20%, 40%, 60% and 80% depth at each set of cell centres (the locations where CFX velocities had been calculated) another short FORTRAN program was written by the author in order to calculate the depth at each point and find the velocities nearest (in the vertical plane) to the elevation corresponding to each depth percentage. The velocity at 20% depth etc, was found as a proximity weighted average of the velocities from the closest cell centre immediately above and below the elevation in question.

Plots of the difference between CFX predicted velocities and site measured values are shown in figures 4.29 to 4.32. The results are summarised in table 4.3. A positive value indicates that the CFX predicted value exceeds the site measured velocity. Again, the comparison suffers from the fact that many of the site measured velocities are one dimensional.

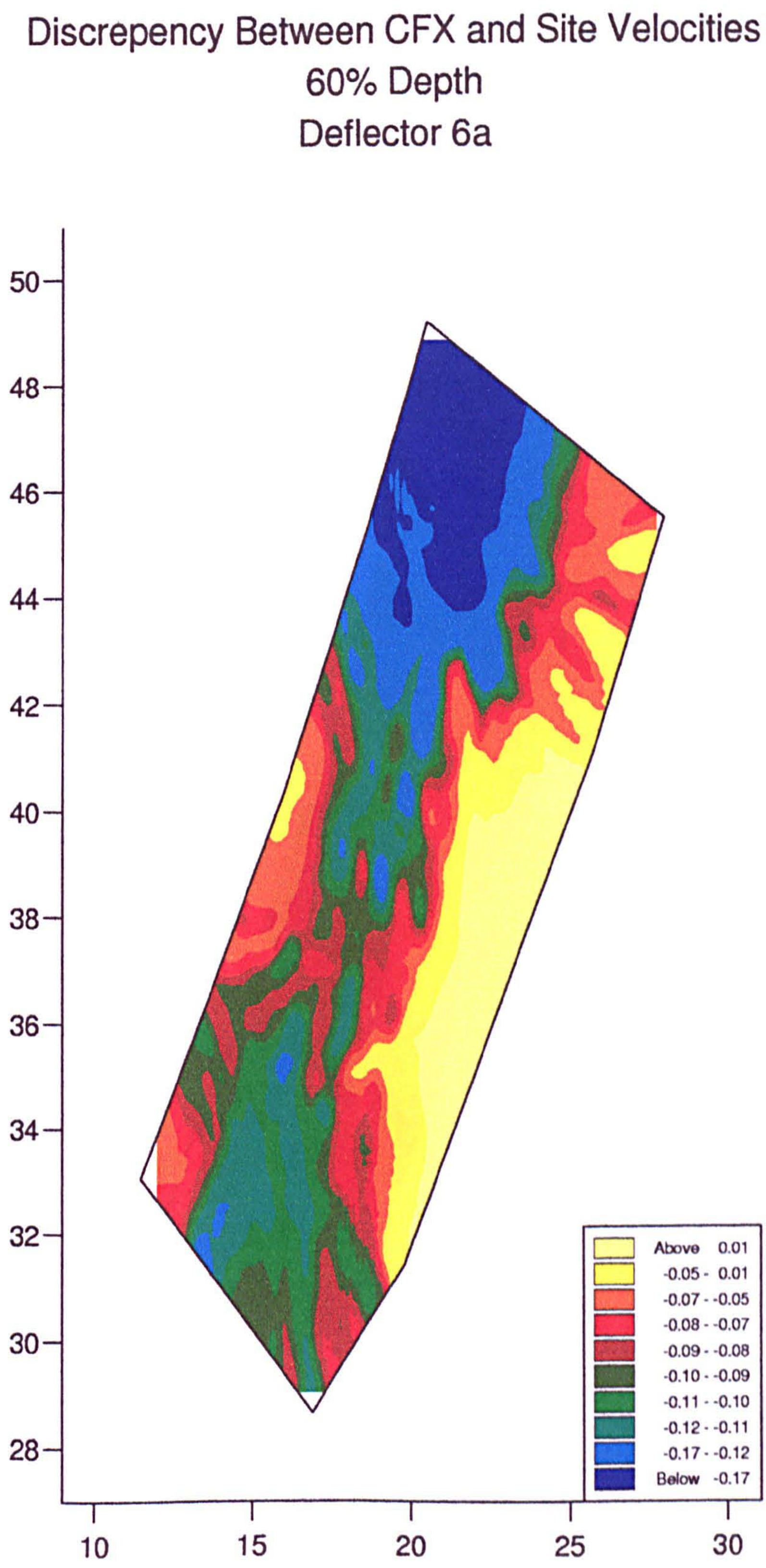


Figure 4.29: Validation of CFX velocities against site measurements for deflector site 6a

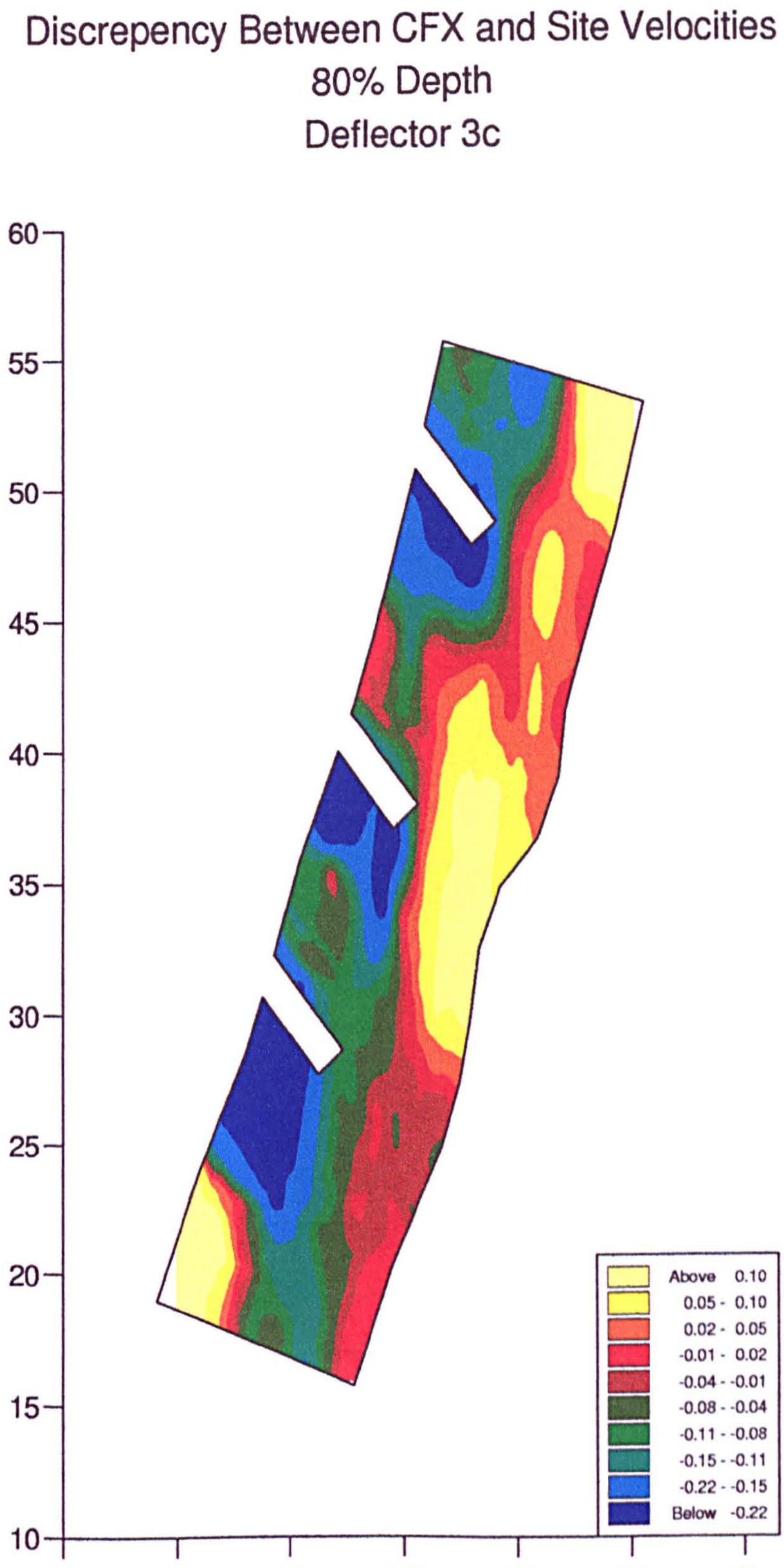


Figure 4.30: Validation of CFX velocities against site measurements for deflector site 3c

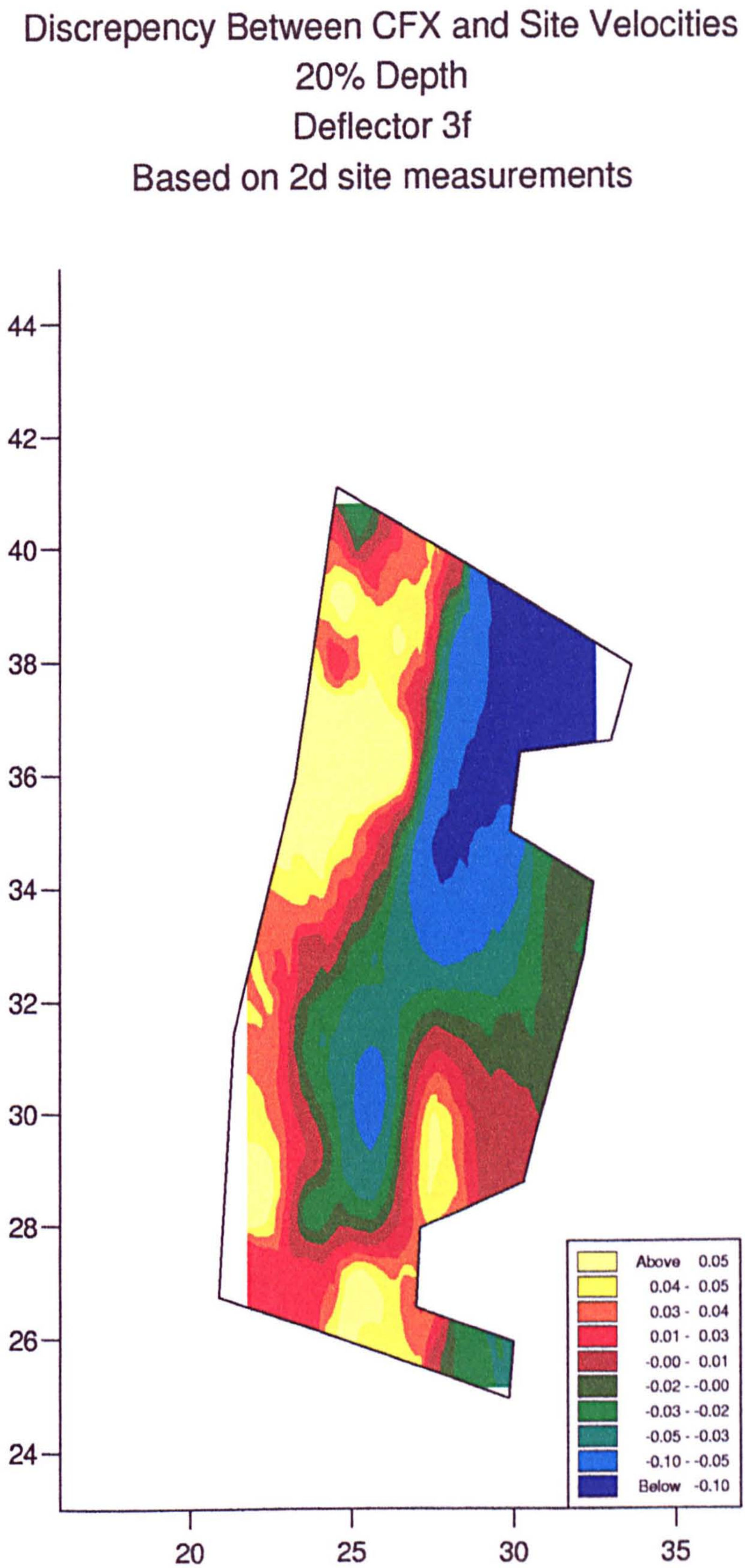


Figure 4.31: Validation of CFX velocities against site measurements for deflector site 3f

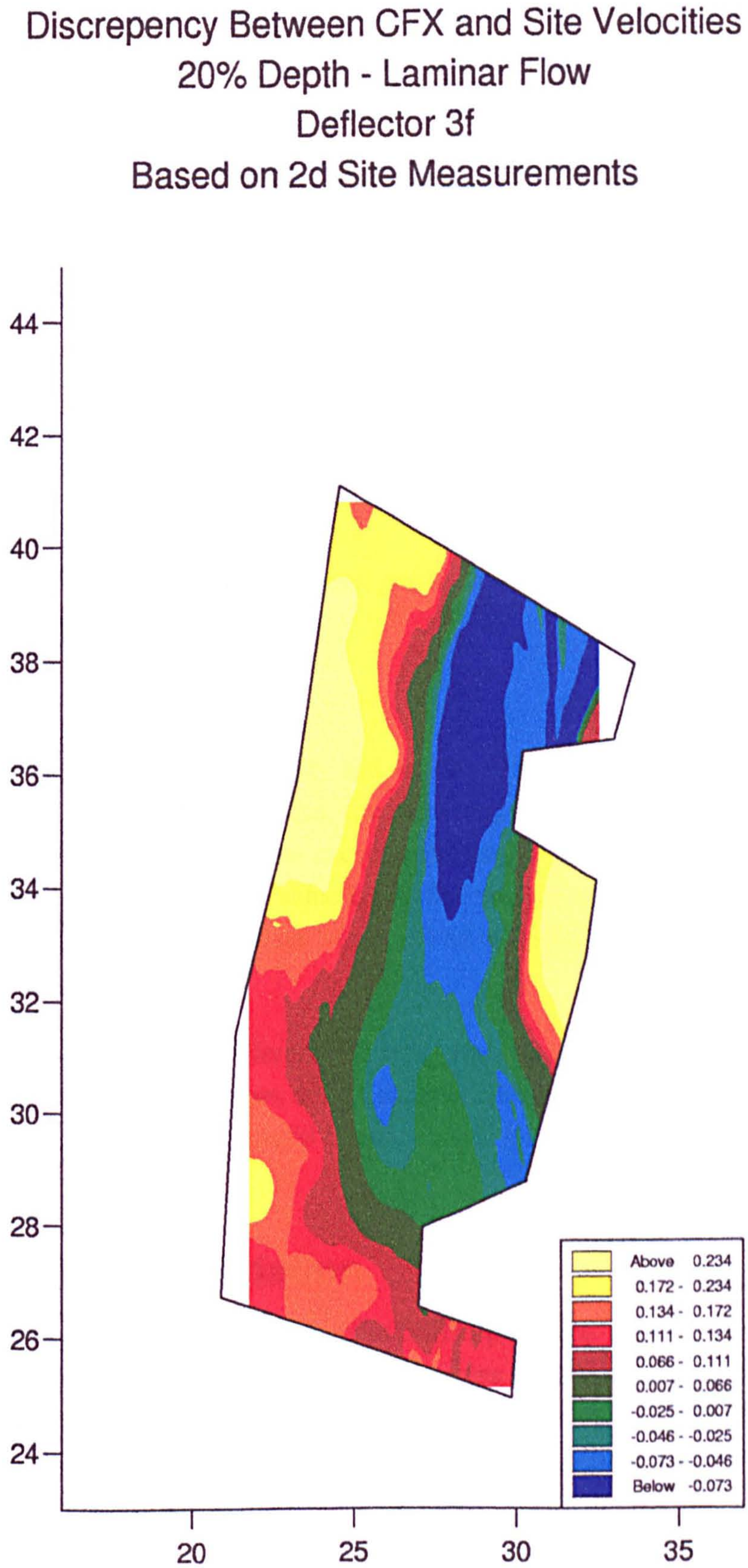


Figure 4.32: Validation of CFX velocities against site measurements for deflector site 3f

4.7. Conclusions

In order to assist in quantifying the accuracy of the two and three dimensional hydraulic modelling software in reproducing the site measured velocities, a percentage based measure was used as shown by equation 4-1:

$$\%diff = \left[(pred_{vel} - site_{vel}) \Big/ \frac{|\,pred_{vel}\,| + |\,site_{vel}\,|}{2} \right] \times 100$$

(4-1) $\%diff$ is the percentage measure, $pred_{vel}$ is the model predicted velocity, $site_{vel}$ is the site measured velocity.

Note: if both the predicted velocity and the site velocity are negative the percentage difference is multiplied by -1. Thus, the percentage difference is always positive if the magnitude of the predicted value exceeds the magnitude of the site value. The method breaks down somewhat if one value is positive and the other negative. In this case the percentage difference will always be calculated as 200% regardless of the magnitude of the velocities.

Quantifying a percentage difference between the two sets of values in this way is not strictly correct as the velocities do not change from one set of values to the other. However, it does provide a useful common basis for examining the accuracy of the results.

4.7.1 SSIIM Validation

Table 4.2 shows what percentage of the SSIIM predicted velocities were within a specified range of the corresponding site measure velocities. At deflector 6a and 3c two comparisons are possible; using the average of the site velocities at 20% and 80% depths at each point, and using just the site velocities measured at 60% depth.

	Less than 10% difference	Less than 20% difference	Less than 50% difference
6a Pre Deflector			
Av. of site vels at 20% and 80% depths	0.0	3.3	10.0
Site vel at 60% depth	5.0	6.6	12.3
3c Post Deflector			
Av. of site vels at 20% and 80% depths	5.7	7.1	15.7
Site vel at 60% depth	8.2	12.7	18.7
3f Post Deflector			
1D Site Velocities – Av. of 20% & 80% depths	10.0	12.5	20.0
2D Site Velocities – Av. of 20% & 80% depths	12.5	15.0	20.0

Table 4.2. Percentage differences between SSIIM and site velocities

Overall, it can be said that the two dimensional results compare considerably less favourably with the site measured velocities than the three dimensional results. This is to be expected, to some extent, as the three dimensional approach solves the momentum equations in all three mutually perpendicular directions, whilst the two dimensional method assumes a hydrostatic pressure distribution leaving only two mutually perpendicular momentum equations (excluding the vertical) to solve. Thus, the three dimensional approach is more representative of the real conditions in the river. This is particularly the case around the deflectors where there can be expected to be considerable vertical velocity components produced as the flow accelerates, due to the rapid asymmetric narrowing of the cross section width. There is also an issue over the representation of roughness in the two models, this is discussed later in the chapter.

The worst agreement between the SSIIM results and the site measurements is at 6a prior to the deflector installation. Here, only 10% or 12.3% of the predicted velocities are within 100% of the site values. The correlation with site measurements is particularly bad in the centre of the channel, where velocities are highest. Here the under prediction of velocities is as high as 0.418 m/s (plot 4.26).

Overall, this result does not match well with the above discussion, which predicts that problems may become most evident with the two dimensional approach where three dimensional effects are prevalent i.e. at the deflectors. However, the quality of the SSIIM results seems to be slightly better with the deflectors present. Between 15.7% and 20.0% of the predicted velocities lie within 100% of the corresponding site values at the post deflector validation sites.

At the deflectors, it is evident from results plots 4.17 to 4.19 plus 4.27 and 4.28 that not only is there a problem with the magnitude of the velocities predicted by the two dimensional code, but parts of the predicted flow pattern are incorrect also. In particular, the recirculation between the deflectors is poorly predicted. The site and CFX predicted velocities show the recirculation occupying the whole of the region between the deflectors (plots 4.21 to 4.24 and 4.25). SSIIM predicts a recirculation within only a portion of the zone between the deflectors for deflector 3f, occupying a space immediately in the lee of the upstream deflector in each case. In the remainder of the zone between the deflectors, the main part of the flow diverges to fill the entire cross section before converging again to accommodate the next downstream defector. For deflector 3c, (plot 4.18) no recirculation is predicted at all between the deflectors..

As with site 6a, SSIIM results appear to be worst where the flow velocities are highest i.e. adjacent to the tips of the deflectors. In validation figures 4.27 and 4.28 the poor relationship between the SSIIM results and the site velocities is shown by the dark blue bands at the deflector tips. Where the velocities are lower, adjacent to the channel boundary or in the recirculation zones, the magnitude of the SSIIM velocity predictions compare more favourably with site measurements.

The most outstanding feature of the results from SSIIM is that virtually all of the predicted velocities are less than the corresponding site measurements. This is suggestive of some problem with the modelling approach used. Despite considerable effort, no obvious cause of the poor quality of the two dimensional results has been discovered. This matter is worthy of further investigation. It is sufficient to say that the depths generated by SSIIM match extremely well with the site measurements, and the backwater profiles generated by ISIS. However, there is obviously some problem in the generation of velocities. The velocities produced by CFX, detailed below, correlate considerably better with the site measurements.

The validation of the SSIIM results is affected to some degree by the algorithm used for this purpose, in the same way as CFX (as discussed in more detail in the next section on CFX validation). The four spatially nearest predicted velocities were used to determine a mean predicted value at each of the locations where site measurements of velocity had been taken. Thus, it is possible that the four constituent predicted velocities are drawn from different parts of the flow structure. This will give rise to a value of the mean predicted velocity which is not truly representative of the predicted velocity at that point in the solution domain. However, similar comments to the application of this method apply to those detailed in the section on the CFX validation. In essence, the method used is considered to be the best available.

4.7.2 CFX Validation

Table 4.3 shows the percentage of CFX predicted velocities found to be within 10%, 20% and 50% of the site values. For example, 77% of the velocities predicted by CFX for deflector 6a were within 50% of the site measured values.

Deflector site 6a shows some of the best correlation between the CFX and site measured velocities. This is to be expected as the scenario being tested is prior to the deflectors being installed and thus the flow structure is less complex. The velocities appear to be considerably better predicted closer to the bed where 67% of the predicted velocities are within 20% of the site values at 80% depth as opposed to only 30% at 20% depth. Figure 4.29 shows that CFX under predicts the velocities in most cases. The results also appear to be considerably worse near to the left bank as compared to the right bank. It is hard to suggest why this might be the case.

The comparison between CFX results and the one dimensional based site velocities at 3c, following the deflector installation, is shown in Figure 4.30. This shows that CFX over predicts the velocities, at the far bank, in the region directly opposite the deflectors. There is also an under prediction of velocities between the deflectors. Overall, the results for deflector 3c are the worst of all with only 53-57% of the CFX predicted values falling with 50% of the corresponding site measured data. This is perhaps indicative of the fact that the comparison is between site velocities measured with a one-dimensional flow meter at a location (i.e. post deflector installation) where there will inevitably be two and three dimensional elements within the flow.

6a Pre Deflector	Less than 10% difference	Less than 20% difference	Less than 50% difference
20% Depth	17	30	77
60% Depth	7	27	83
80% Depth	37	67	86
3c Post Deflectors			
20% Depth	13	29	56
60% Depth	16	30	53
80% Depth	24	32	57
3f Post Deflectors:			
1D Measurements			
20% Depth	13	37	71
40% Depth	30	51	70
80% Depth	22	44	64
2D Site Measurements			
20% Depth	30	60	85
40% Depth	18	60	85
80% Depth	10	18	55
Laminar Flow, 2d Site Measurements:			
20% Depth	10	27.5	72.5
40% Depth	12.5	25.0	67.5
80% Depth	5.0	12.5	25.0

Table 4.3. Percentage differences between CFX and site velocities

Because the above validation process, for deflector 3f and 3c, is entirely based on site velocities taken with a one dimensional flow meter it is not given much extra consideration. Clearly, considerable lateral velocities do exist in the region around the deflectors. These velocities have not been measured and are not included in the site velocities. As a result, the discussion will focus on the two-dimensional site velocities taken at deflector 3f.

The two dimensional site measured velocities for deflector 3f, following the deflector installation, show overall better agreement with the CFX predicted velocities than all of the other cases. This demonstrates the importance of measuring both the lateral as well as the streamwise velocities. 85% of the CFX predicted velocities are within 50% of the site values at 20% and 40% depth, although only 55% are at 80% depth. This may suggest that the boundary roughness is not being correctly represented in the CFX model in some way. Further away from the boundary, the effect of the roughness becomes more distant and this may explain why velocities here are better predicted.

The contour plot of the difference between the CFX and site velocities, Figure 4.31, shows a different pattern to those based on one dimensional site velocities. Here the predicted velocities between the deflectors match much better with the site values. There is a region where the CFX values over predict the site velocities, at the opposite bank from the deflectors, but in two out of the three cases the over prediction is less than 0.07m/s. At 80% depth the over prediction by CFX is as high as 0.137m/s which is very considerable and again suggests there may be some problem with the boundary roughness.

The region where the main under prediction of velocities takes place is immediately adjacent to the tip of the deflectors (particularly the second deflector) and extending downstream (to the downstream limit at which site velocities were taken in the case of the second deflector). This under prediction of velocities is considerably greater in magnitude than any over prediction at 20% and 40% depth. At 80% depth it is overshadowed by the magnitude of the over prediction of velocities at the far bank opposite the deflectors.

In general terms, comparing the predicted velocity vector plot at 20% depth, figure 4.21, with the corresponding two dimensional site measurements, Figure 4.25, CFX seems to reproduce the pattern evident in the site data quite well. The region of greatest velocity develops just off the upstream end of the upstream deflector and moves into the left side of the channel at an angle of about 20 degrees to the flat end of the deflector. This plume of high velocity begins to develop about 3.5m upstream from the first deflector as the flow angles in from the right bank towards the deflector tip. A zone of weak recirculation is left upstream of the first deflector. A reverse circulation is also evident between the two deflectors and downstream of the second deflector.

The reverse circulation between the two deflectors can be seen to occupy the whole of the space between the two structures, and in fact appears to 'bulge out' into the main part of the channel by about 1.5m. The current reattaches itself to the bank about 6m downstream of the second deflector. All of the effects outlined can be seen clearly in the CFX plots and are confirmed, to varying extents, by the site data. It is unfortunate that considerably more two dimensional flow measurements were not taken in the regions upstream and downstream of the deflectors at 3f, in greater spatial detail around the deflectors, and at the other sites. This would have allowed for a much more complete analysis of the accuracy of the flow structure predicted by CFX. Full descriptions of the flow patterns predicted by CFX and the site measurements are not repeated for 40% and 80% depths as they are sufficiently similar to those discussed above.

Overall it is difficult to comment on how good the agreement is between CFX and site velocities is. The way that boundary roughness is incorporated in the CFX model has already been mentioned as a potential source of error. The way in which roughness is incorporated in three dimensional models is a rather specialised area. The method followed here is outlined in chapter 3 and merely followed a procedure recommended in an earlier piece of work. This may require further examination. However, the fact that CFX predicts velocities that are both greater than and less than the site values at 80% depth suggests that this may not be the only source of error.

Figure 4.22 shows the velocities predicted by CFX at 20% depth using laminar flow, i.e. without the use of a turbulence model. Figure 4.32 shows the comparison between these predicted velocities and the site measurements. The velocities that are predicted when the model is run without a turbulence model are considerably higher than those predicted using a turbulence model. Figure 4.32 shows that the over prediction of velocity, compared to site measurements, is as high as 0.234 m/s at deflector 3f. With the turbulence model running, the maximum over prediction of velocity is only 0.05 m/s (figure 4.31). This result is confirmed in Table 4.3 which shows that the percentages of the predicted velocities falling within 10, 20 and 50% of the site measurements has declined considerably with the removal of the turbulence model.

The predicted flow pattern is also affected by the lack of a turbulence model. Figure 4.22 shows that the recirculation upstream of the first deflector, evident in the site measurements, is not predicted at all in the laminar solution. However, the main recirculation, between the two deflectors is still present and largely unaffected. The recirculation behind the second deflector is also still present. The extent to which the results are affected by running a laminar problem shows the importance of the accurate modelling of turbulence in the solution.

The increase in velocities found on removing the turbulence model is to be expected, as the effect of turbulence on the time averaged properties of fluid flow is similar to adding viscous stress. This creates an additional energy loss that slows the flow down. By simply removing turbulence from the computation, the extra viscous stress is lost and the flow moves, consequently, more rapidly.

It should be noted that it is not possible to run a laminar problem with a logarithmic wall function in CFX. This is because the properties of the turbulence are used in the calculation of the logarithmic wall function, as discussed in chapter 3. As a result, it was necessary to use a more simplistic quadratic wall function to obtain the laminar solution.

One other factor, which may explain some of the inaccuracy of the CFX solutions, is the fixed lid assumption discussed in chapter 3. Although each solution was checked to ensure pressures at the water surface were not excessive, there will inevitably be an error introduced if any gauge pressure (other than zero) exists at this level. The complex flow structure at the deflector (and in the vicinity of any bend) will inevitably produce variations in the elevation of the water surface across the channel width. These have not been taken into account. In addition, the fact that the transport of material is not taken into account may have some affect. The Idle suffers from large volumes of wind blown sediment. Transporting this material must take some energy out of the flow, but this complex phenomenon has not been included in the model.

A further problem lies in the algorithm written by the author which utilises the four spatially nearest CFX velocities to each site velocity, in order to determine a mean predicted value at each location that a site measurement was taken. This was necessary as the centres of the CFX flow cells (where each of the flow variables are calculated) do not coincide with the locations of the site measured velocities. Unfortunately, this may introduce some errors, particularly in areas where the flow is rapidly changing, for example in the region of flow separation at the deflector tip. This has already been noted as a point where CFX under predicts the flow speed. In this region, examination of the four velocities used in the determination of each of the mean velocities shows, in a small number of cases, that some of the velocities are positive and some are negative. In some other places the four velocities are of quite dissimilar magnitudes. This means that the velocities are being taken from very different parts of the flow structure. The mean will, therefore, not be a very accurate reflection of the velocity at that point. An alternative would have been to use the single spatially nearest CFX velocity to each site measured velocity. However, this technique may have similar problems in that the nearest CFX velocity may again be in a different part of the flow structure. Increasing the number of cells in the solution would be expected to improve the accuracy of this validation procedure, however it would lead to larger simulation times and larger solution files.

One final possibility considered was to overlay the locations at which site measurements had been taken onto a colour contour plot of the CFX velocities at 20%, 40% and 80% depth. The predicted CFX velocities could then have been read off at the relevant points using the colour coded key. This may have been the most accurate technique, however it was not found to be possible to either import the locations of the cross sections from AutoCAD into Gsharp, or to take the colour contour plot from Gsharp and import it into AutoCAD. It would have been possible to print the Gsharp contour plot and overlay the locations of the site measurement locations on by hand. However, this was felt to have considerable limitations regarding the potential of locating the site measurement points on the plot with sufficient accuracy. In particular, the fact that Gsharp is a package for representing results in a presentation like format (rather than in an engineering way at a particular scale) limited the potential accuracy. A minor error in locating the measurement point on the plot where the flow structure was changing rapidly (for example in the separation region) would produce a very different velocity. In addition, it is only possible to determine approximately ten distinct colours from the plot key and thus the velocities read off at the site measurement locations would be limited to ten values.

The potential fallibility of the CFX velocity predictions have been discussed, but it should also be borne in mind that site measurements of velocity, taken with electronic flow meters, are also subject to potential inaccuracy. In fact, some research suggests that very careful calibration of equipment is required or considerable errors can be introduced into measurements (Bowles et al, 1998). As a result, the fact that discrepancies exist between site and simulated velocities should not be automatically attributed to failings within the software used, or the methods used to manipulate the predicted values.

4.8. Discussion

The analysis and results presented here are useful in indicating the potential application of one, two and three dimensional modelling. One dimensional modelling is of great value in predicting the effect of rehabilitation measures on flood peaks. It provides all of the information that would be required in the design of a traditional flood alleviation scheme. In this case, it can be seen that the effect of the deflectors on river levels is not very considerable.

Inevitably though, one dimensional models are not sufficient if the modeller is interested in the detail of the effect of a design on flow variability. In this case, a two or three dimensional model is of great use in describing the effect of the installation of alternate designs on the flow field. If the intention of a design is to increase flow variability, the potential effect of alternate proposals can only be fully tested with a two or three dimensional modelling approach. In later chapters, the potential advantages of using a three dimensional model in describing the movement of sediments, and in analysing aquatic habitat are explored.

The validation of the predicted velocities from SSIIM is not particularly encouraging, showing some significant discrepancies between the predicted and site measured velocities. The major problem lying in the under prediction of velocities and the prediction of the recirculation patterns downstream and between the deflectors. The problem may be in part due to the assumption of a hydrostatic pressure distribution in the two dimensional code. At any bend in a channel, the momentum of the flow carries a secondary current of water at the surface towards the outside of the bend. This creates a superelevation of the water surface at the outside of the bend which forces a second cross wise velocity into existence, near the bed, moving towards the inside of the bend. This effect is shown in Figure 4.33. CFX can reproduce this velocity pattern but SSIIM, with its hydrostatic pressure distribution, cannot. As a result of SSIIM's inability to predict the vertical velocities in the helical flow pattern, which will be produced at the deflectors, (a deflector effectively creates a bend within the flow) it predicts a somewhat different flow pattern which matches less well with site observations.

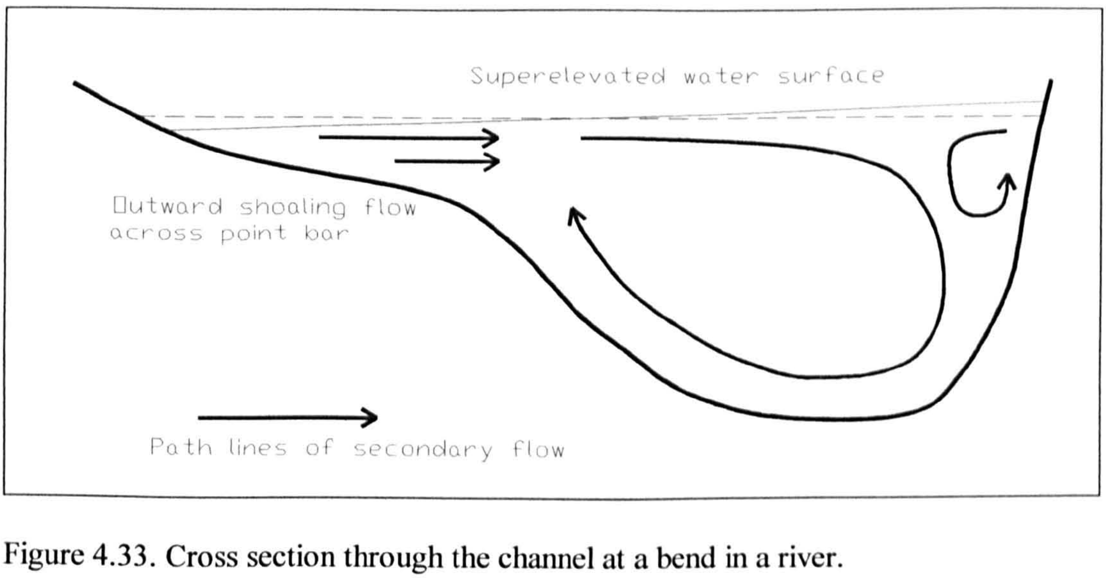


Figure 4.33. Cross section through the channel at a bend in a river.

A further possible cause of the poor results from SSIIM may be the *k-e* turbulence model employed. One problem with this is that the *k* term is dependant on the velocities in the three mutually perpendicular directions, as shown by equation (3-19) in chapter 3. Since the vertical velocities in a two dimensional model are solely based on a hydrostatic pressure assumption, the calculated value of *k* would not be the same in two and three dimensional models. Thus, the calculation of turbulence properties should have greater accuracy in a three dimensional model..

In addition, in SSIIM the *k-e* turbulence model is employed alongside the Manning's n value. The Manning value is used in SSIIM purely to represent boundary roughness. However, the Manning's n value is in effect a form of turbulence model in itself. In one dimensional models, there is no turbulence model so Manning's n effectively accounts for friction loss and energy loss due to turbulence. In SSIIM, energy loss due to turbulence is calculated by the *k-e* model, but the same value of Manning's n is being employed to represent boundary friction as in the one dimensional model. Therefore, it would appear that the effect of turbulence may be accounted for twice. A lower value of Manning's n was tested within SSIIM but the observed backwater profile then differed from the one dimensional simulated depths, and those measured on site. Thus it was still necessary to employ a Manning's n of 0.085 in the two dimensional model to ensure that depths were accurately replicated.

In CFX, the Manning's n value of 0.085 was used to generate the backwater profile that determined the position of the fixed lid. However, the wall functions that were used to set the boundary roughness were based upon sediment samples taken on site at several points across the channel width, and at a number of different cross sections within the river. These samples were taken by students of the School of Geography. Analysis of the sediment samples taken on site revealed the D_{84} value to be 0.66mm. Thus the k_s value, which is equivalent to $3.5D_{84}$ (Hodkinson, 1996), is $2.31\text{x}10^{-3}$ m. The procedure for the setting of the wall function values, based on k_s, is set out in the section on three dimensional modelling in chapter 3.

A number of formulas are available which relate Manning's n to the effective roughness k_s. One such formula is suggested in equation (4-2) (Massey, 1968)

$$n = \frac{0.0564m^{1/6}}{\log_{10}(14.86m/k_s)}$$

(4-2) Massey's formula relating Manning's n to the effective roughness k_s. m is the hydraulic mean depth (A/P)

Equation (4-3) gives an alternative formula (Chanson, 1999)

$$\frac{1}{n}\left(\frac{m}{4}\right)^{1/6} = 17.7\log_{10}\left(\frac{m}{k_s}\right) + 10.1$$

(4-3) Chanson's relationship between Manning's n and effective roughness.

Other authors have suggested formulae relating Manning's n directly to the grain sediment size, as oppose to the effective roughness. Examples are the relations of Strickler (1923) and Henderson (1966) which are given in equations (4-4) and (4-5) respectively:

$$n = 0.041D_{50}^{1/6}$$

(4-4) Stricker's relationship between Manning's n and sediment diameter

$$n = 0.038D_{75}^{1/6}$$

(4-5) Henderson's relationship between Manning's n and sediment diameter

The existence of a number of different formulae for the relation between Manning's n and boundary roughness show that the correlation between the two is not clear cut. This is to be expected as Manning's n is an empirical value encapsulating not just roughness relating to particle size, but also form roughness which can be due to the presence of bed formations or sinuosity. As has been explained, in the three dimensional work carried out in this thesis a Manning's n value was used to determine the water surface elevation, whilst a separate particle roughness, based on sediment sampling, was used to set the wall functions. The Manning's n utilised was the quoted value given by the NRA used to validate their one dimensional model for the Idle. Thus the two values (Manning's n and k_s) were derived from separate equally valid sources. The possibility of working either from Manning's n to determine a k_s value, or vice versa, was considered. It was decided that it was better to base the computation of the backwater profile on the global value of Manning *n* quoted by the NRA, which took into account wider scale roughness factors. At the same time, the value of k_s could be determined from more site specific sediment analysis. Given that there are multiple equations available relating Manning's n to particle roughness, the method described was considered to be the best available.

Formulae (4-2) to (4-5) suggest that the k_s value of $2.31 x 10^{-3}$ m represents a smoother value of wall friction than the Manning's n value of 0.085. This suggests that there may be some discrepancy in using the two different values to represent wall friction. However, as has been discussed, both values arise from separate valid sources.

With SSIIM it is possible to input a value of k_s to set the wall roughness and a separate value of Manning's n for the computation of the backwater profile. However, in that case the final solution is the same as setting a single value of Manning's n which is equivalent to the k_s value. In other words, allthough it is possible to set a Manning's n value of 0.085 and a k_s value of 2.31x10-3 in the control file, the final solution is only based on the k_s value. Thus it was necessary to use a Manning's n value of 0.085 with SSIIM in order to ensure that the backwater profile is correct. Since equations (4-2) to (4-5) suggest that the Manning *n* value of 0.085 is a rougher value than a k_s value of 2.31x10-3, this may explain in part why the SSIIM predicted velocities are lower than both the CFX and site measured value.

As a result of the factors outlined above, there may exist problems in applying SSIIM, and two dimensional models in general, to predict velocities following a rehabilitation that involves the construction of flow deflectors. Where a rehabilitation entails less dramatic changes to the river (for example the reprofiling of some bends), which is likely to produce a less complex flow structure, SSIIM may be sufficiently detailed to predict the post scheme velocities accurately. This assumption would need to be tested. Other two dimensional codes, such as Telemac2D, may predict velocities where the flow structure is complex with more accuracy. However, the results from this thesis indicate that a three dimensional model is to be preferred to the use of a two dimensional model for the prediction of post scheme velocities, at the design stage.

Unfortunately, three dimensional modelling is unlikely to become a widely used tool by consulting civil engineers until a package becomes available which is more tailored towards the needs of the river modelling community. This would require the user to have considerably less knowledge of computational fluid dynamics than CFX requires. Creating geometry files with CFX is a time consuming and tedious process. In addition, to instruct the program to output the predicted velocities (and their associated co-ordinates) to an ASCII file, requires the user to write a FORTRAN subroutine. To create a command file requires the user to determine which effects (for example buoyancy, coriolis effect and enthalpy) are considered to be important, and need including, and which are not. These criticisms of CFX are to be expected as the package is designed to be generally applicable to a wide variety of three dimensional flow problems (including gas dynamics, wind flows etc). A user friendly three dimensional package is required which is specifically aimed at the river modelling community.

Chapter 5

River Idle Habitat Modelling

5.1. Introduction

The days when large river engineering schemes could be implemented without regard to their effect on the aquatic environment are gone. Increasing public awareness, and concern over environmental matters, means that it is necessary to be able to predict the effect of river engineering works on aquatic species. In addition, rehabilitation and restoration schemes which enhance the aquatic environment are more justifiable if the resulting habitat improvement can be quantified at the feasibility or design stages. This has given rise to the development of a number of competing hydro-ecological models. The purpose of these models is to relate physical changes to the watercourse, or alterations in the flow regime, to a measurable effect on the abundance of aquatic species.

This chapter begins with a review of current methods for the modelling of watercourses in terms of the habitat requirements of the aquatic organisms present. This is often referred to as instream flow modelling. The techniques outlined here are predominantly focused on the assessment of fish habitats. Relatively little attention is given to assessing invertebrate habitat, despite the fact that they form an important part of a river's ecosystem. The reason for this approach is that difficulties in the sampling of invertebrates (due to the large amount of organisms collected) leads to difficulties in model validation and verification. In addition, there can be problems with the taxonomic identification of invertebrates, and difficulties in assigning benefit to schemes which improve habitat specifically for them. Most regulatory authorities make the assumption that the provision of an adequate flow regime for fish will be sufficient for invertebrates. However, Gore et al (1998) has pointed out, with reference to the work of Statzner et al, that lotic insect species do in fact exhibit a narrower range of flow tolerances than their co-existing fish species.

There are a number of different packages available for the analysis of aquatic habitat. Consideration here is limited to a few key ones that highlight a number of different techniques for assessing hydroecology. The first approach discussed is the Instream Flow Incremental Methodology (IFIM) and its associated software PHABSIM. This is the most widely used, discussed and criticised approach available. It also forms the basis for the work presented later in the chapter. The approach seeks to bring together the twin concepts of individual species preference curves and hydraulic modelling. Predicted depths and velocities are compared against the habitat requirements of a number of target species in order to determine an available area of habitat for each species. The criticisms of IFIM are discussed, and a number of previous applications of the method are outlined.

Perhaps the biggest rival to PHABSIM is RCHARC. This represents a much simpler approach to habitat modelling. It essentially seeks to compare depths and velocities between a restored reach of river (or predicted depths and velocities in a planned restoration) against those in an adjacent 'ideal' reach. The concept behind RCHARC is that flow variability and habitat are linked. Thus if the variability of depths and velocities in the restored reach approach that in the 'ideal', the restoration should produce the same quality of habitat.

SERCON and RIVPACS both utilise ecological databases to compare the abundance of species in a reach of river under consideration with what has been previously observed in similar circumstances elsewhere. Any discrepancy would suggest the presence of environmental stresses that could require further investigation.

CASIMIR is essentially an improvement on the PHABSIM approach. It estimates available habitat for fish, invertebrates and vegetation. Fish habitat is evaluated using the same approach as PHABSIM, i.e. suitability curves. Invertebrate habitat is related to calculated values for bed shear stress, and the likely abundance of vegetation on the flood plain is related to depth and frequency of flooding.

Advances in two and three dimensional modelling techniques, together with the increasing availability of cheap high powered computing, means that spatially explicit habitat modelling is now possible. However, habitat modelling is still a relatively new field and much research is still required into the biology of the aquatic environment before full advantage can be taken of these new techniques. The potential advantages of two and three dimensional modelling approaches to aquatic habitat studies are discussed later in this chapter.

The second part of this chapter looks at the habitat modelling carried out to assess the effect of the rehabilitation of the River Idle. Improving aquatic habitat was the central purpose of rehabilitating the River Idle, so the quantitative assessment of the habitat improvement detailed here gives some indication of the likely success of the scheme. Three FORTRAN programs were written from scratch, for the purposes of this thesis, and used to assess the improvement in habitat. The programs, and the results that were produced using them, are detailed. Some discussion of the results obtained from the programs, and the limitations in the approach used are given at the end of the chapter.

Unfortunately, it is not possible to validate the results from the habitat modelling in any way. This is mainly due to the fact that the programs yield predictions of available habitat area, and not fish biomass. The reason for this is that there are too many complicating factors to be able to predict fish numbers. Factors such as water quality, the degree of predation and competition for food all play a part in fish habitats but these factors cannot yet be quantified and included in habitat models. In addition there will inevitably be a lag between the time a habitat is improved, and the time at which it is colonised by fish species. The exact duration of this time lag is hard to determine with any confidence. More research is needed into this field and this is discussed later in the chapter, and in the general conclusions to this thesis, chapter 7. No fish surveys have been carried out since the construction of the rehabilitation scheme. As a result, it is not possible to determine, as yet, the extent of any habitat improvement that has occurred.

Despite the fact that the results from this habitat modelling exercise cannot be validated, they do highlight the techniques used and the results that can be obtained using current methods. The results are useful in indicating the likely effect of the deflectors on different fish species. In addition, the greater accuracy that can be achieved with a two or three dimensional approach can clearly be seen in the results.

5.2. Instream Flow Incremental Methodology (IFIM)

The Instream Flow Incremental Methodology (IFIM) provides a method of quantifying the impact of any engineering scheme, which influences discharge or flow pattern, on the aquatic habitat. The method was introduced in 1974 by the Aquatic Systems Branch of the U.S. Fish and Wildlife Service. It is based on the premise that aquatic species (fish and invertebrates) exhibit discernible preferences for specific physical habitat variables which are dependant on the river discharge. Certain target species are selected, and the habitat requirements of these species are quantified in terms of flow velocity, water depth, substrate character and available cover Bullock, Gustard and Grainger (1991). It is then possible to determine the available habitat for each species at any specified discharge. This allows the production of a plot of available habitat area versus discharge relationship for each target species. Using a computer model it is possible to simulate the effects of any new scheme on the river depths, velocities and substrate, and hence produce another available habitat versus discharge relationship for the post scheme situation. A comparison of the two plots gives an indication of the effect of the design proposals on the aquatic habitat.

5.2.1 The IFIM Concept

IFIM is essentially a concept or procedure which enables the estimation of an available habitat area for several target species based on their preferences for a range of habitat conditions. This is distinct from PHABSIM which is a piece of software which implements the IFIM procedure

The IFIM concept is based on the fundamental principle that individual species habitat requirements can be quantified in terms of a small number of habitat variables, and that these preferences for certain types of habitat can be expressed in terms of a suitability index. Species will respond to changes in the aquatic environment by relocating to areas where their preference for the hydraulic variables is higher. However, they will continue to use areas with a lower suitability index, but with a correspondingly reduced frequency.

IFIM is a tried and tested approach and is applicable to both coarse and game fish Merle and Eon (1996).

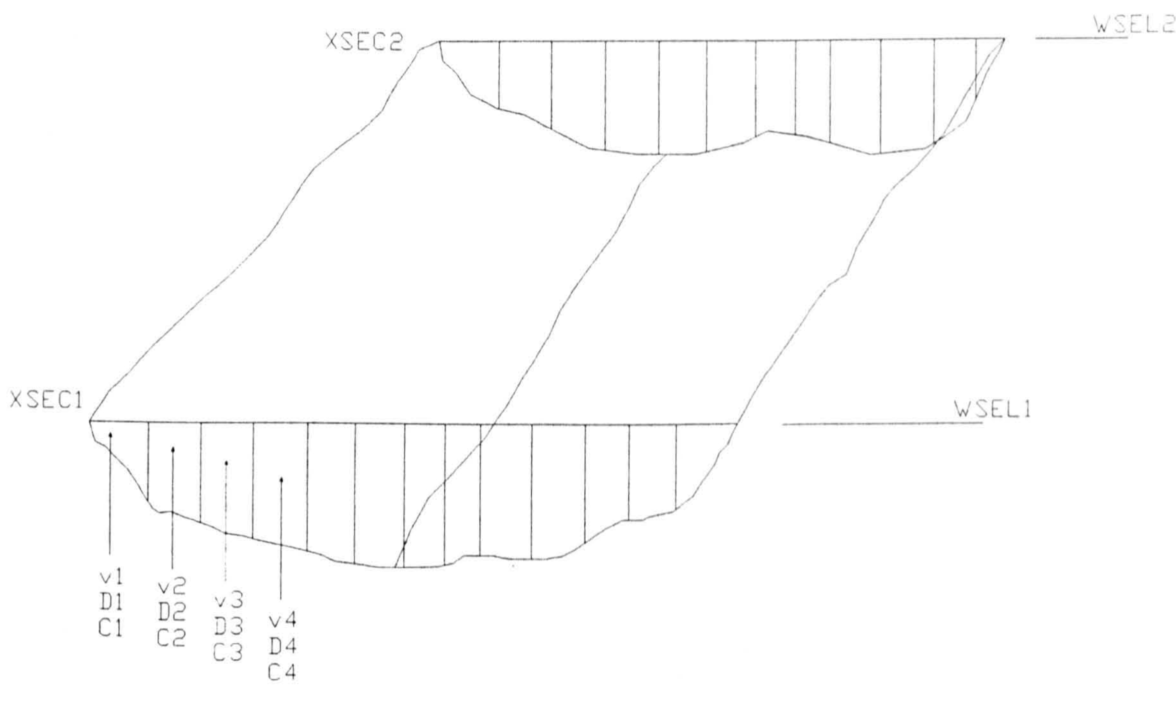


Figure 5.1: Cross section data required for input to PHABSIM

5.2.2 PHABSIM

PHABSIM is the standard piece of software that is used to implement IFIM. There is a proscribed procedure to follow in order to carry out a study using PHABSIM. The various stages are outlined below.

5.2.3 Determination of Hydraulic Variables

The first part of the habitat modelling procedure is to determine velocities and depths for each flow cell for a specific discharge. PHABSIM contains several one dimensional hydraulic modelling subroutines for this purpose, namely IFG4, MANSQ and WSP. IFG4 and MANSQ use stage-discharge relationships at each cross section (based on surveyed flow data) to evaluate water depths, and Mannings equation to determine velocities for each cell across the transect width. An adjustment to the calculated values is carried out to ensure mass balance. WSP uses a step backwater model to simulate water levels at each transect. The water levels are then input to IFG4 to determine cell velocities. In addition, each cell has a cover or substrate index value assigned by the user, based on the bed material present or the degree of shade. The result of this is as shown in figure 5.1. Each cell, which forms a portion of the width of the river between two cross sections (XSEC1 and XSEC2), has a calculated velocity v, a depth D and a substrate/cover value C, for the specific discharge under consideration.

The mean column velocity that is calculated, for each cell, during the hydraulic modelling stage is not necessarily the value that is used in later stages of the program. Instead, a vertical velocity distribution can be assumed in each cell, based on one of the equations in chapter 3. Each of the target species is free to move vertically within each cell to a position where its nose velocity is most suitable. This nose velocity is then used in subsequent calculations.

5.2.4 Species Preference Curves

Once the above hydraulic variables have been calculated, the next stage is to combine them with each species habitat preferences in order to determine a measure of the habitat suitability of the reach of river. The part of the software that carries out this part of the calculation is called HABTAT. The extent to which each of the target species exhibits a predilection for each of the hydraulic variables is expressed in preference curves figure 5.2. These consist of a numerical representation of a suitability index value (0 = least suitable 1 = most suitable) on the vertical axis, against each of the hydraulic variable on the x axis. Note that it is necessary to include not only different species of fish, but also different life stages in order to examine the effect of any design on a target species' whole life cycle.

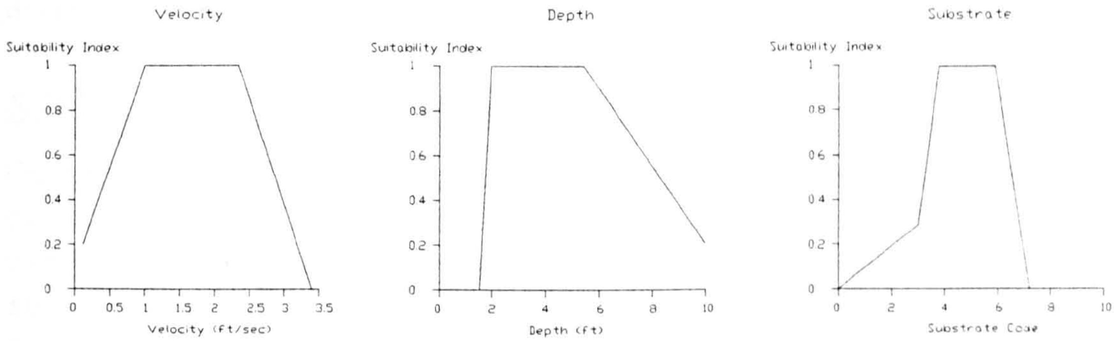


Figure 5.2 : Preference curves for Adult and Juvenile Chub from Bullock et al (1991)

5.2.5 Derivation of Preference Curves

Species preference curves derive from two main sources. One is through the use of field observation of the target species. A survey is carried out, commonly using either divers or electrofishing, to establish the hydraulic conditions in which the species are more commonly found to be resident. The observed frequency of occurrence of the target species, in a variety of hydraulic conditions, allows the calculation of a suitability index plot for each of the hydraulic variables. The problem with this approach is that the data is limited to the range of the hydraulic variables at the time that the survey was carried out. The actual type of habitat that the target species would exhibit the highest preference for may not be present at all, or only be present in a small quantity. Therefore, to improve on this approach some researchers prefer to include correction factors to allow for the fact that some of the hydraulic variables were present in limited quantities during the survey. The other potential source of information is via published studies, or from professional experience.

5.2.6 Combining Suitability Indices

HABTAT produces habitat suitability index values, between 0 and 1, for each of the three hydraulic variables calculated earlier in the software, and based on the individual species preference curves. A variety of options are then available to combine the three habitat suitability indices to produce one, overall, suitability value per cell. Either the three suitability values are multiplied together, or a geometric mean is calculated, or the minimum of the three values can be taken, or alternatively the user may specify an entirely separate function to generate an overall value. The multiply option suggests that the optimum habitat occurs only if all three of the original indices are high. This is the most commonly used method. The geometric mean suggests that if two of the variables are high then the third will have little effect unless its value is very low. The minimum method suggests that the habitat is only as good as the worst of the component indices. The three methods can produce widely varying values for the overall habitat index value.

5.2.7 Weighted Usable Area

Once the overall habitat index value for each cell has been generated, it is possible to calculate a Weighted Usable Area (WUA) of habitat for the site in question by multiplying each cells value by the plan area of the cell, and then summing the totals. The WUA represents an area of available habitat for a target species at the particular discharge being considered. At a different discharge, PHABSIM will calculate different velocities and depths in each cell, and the plan area of each cell will also alter. Consequently, a different WUA value will be calculated.

It should be noted that WUA is a measure of habitat area available to a particular fish species. It is not a prediction of fish biomass in a particular reach, at a particular discharge. The reason for this is that there are too many complicating factors, as referred to in the later section on the criticisms of IFIM and PHABSIM, to be able to relate a particular set of hydraulic variables to an amount of biomass. The aquatic environment is too complex, and the range of factors influencing fish behaviour is too vast to make this relation. WUA does, however, provide a means by which to assess the amount of available habitat, and to make some assessment of the affect of any changes on fish species. Some researchers have carried out field sampling to assess the relationship between WUA predicted by PHABSIM, and actual fish biomass. Boudreau et al (1996) reports that Capra, Nehring and Anderson, Jowett et al, Bovee et al, Gowan, Orth and Maughan have all found good correlation. On the other hand, Conder and Annear, Scott and Shirvell, and Shirvell et al did not find any relationship. In a study of benthic species on the West Sandy River (West Tennesee, U.S.A.), PHABSIM was used to predict the improvement in habitat resulting from the installation of two artificial riffles composed of large cobbles and boulders. Comparing results on a cell by cell basis, excellent correlation was observed between PHABSIM predictions and observed community diversity Gore et al (1998).

The general procedure with PHABSIM is not to model the entire reach of river under consideration, but to select several sites that together are representative of the habitat encountered in the whole of the area under consideration. The results from the individual sites that have been modelled can then be summed, using some method of weighting based on the frequency of occurrence of each site, to give a WUA for the entire river reach. It is possible to calculate WUA at a range of discharges and, hence, generate a plot of WUA versus discharge for each of the life stages of each of the target species. This method can be particularly useful in the consideration of the necessary releases from reservoirs, in order to maintain a minimum flow for the aquatic species in the river downstream. It is possible to see the loss or gain in habitat with varying discharge. The optimal discharge, to maximise the available habitat, can also be determined for each species.

5.3. Criticism of IFIM and PHABSIM

5.3.1 Interrelation of Variables

There are many criticisms of both the IFIM concept, and the PHABSIM software. The first is that in the calculation of individual habitat suitability index values, depth, velocity and substrate are treated as independent variables. Gore and Nestler (1988) point to the criticism provided by Patten and Mathur et al on this point. In reality, these factors are known to be strongly interrelated. The type of substrate influences the flow resistance, and hence the depth and velocity. The resulting depth and velocity determine the bed shear stress and the sediment transport capacity, which themselves contribute to the type of material that accumulates, or is exposed, on the riverbed. Therefore, to treat each of the variables as independent is obviously incorrect. The development of multivariate suitability functions that express the interaction of the physical variables may provide a significant improvement to the methodology. Bird (1996) notes the work done in this area by Orth and Maughan, Voos and Bullock.

5.3.2 Relative Importance of Variables

There is no universal agreement over the relative importance of the various factors in IFIM. The amount of cover that is available is known to be a significant factor, and PHABSIM allows a suitability index value to be determined within the WUA calculation. However, Bird (1996) refers to the work of Hartman, Dolloff and Bugert in observing that this factor is often excluded from calculations due to modelling difficulties produced by its many functions and interactions with other variables.

5.3.3 Species Preference Curves

5.3.3.1 Derivation

Some of the main criticisms of IFIM arise out of its dependence on the concept of species preference curves. The first criticism of this area derives from the means by which the curves themselves are determined, which is commonly form some form of fish survey. This generally involves either electrofishing, or by direct observation using snorkelling.

With electrofishing, an electric current is passed through the water and all fish in the immediate area are temporarily stunned. The fish float to the surface allowing their number to be assessed. The main problems with this type of survey are that individuals can be carried away from the actual habitat that they occupied before they are discovered. Gore and Nestler (1988) refer to the work of Bain et al to demonstrate this. Also, high turbidity can make it difficult for the survey crew to see the fish. The main problems with fish surveys using snorkelling again arise out of poor visibility, and the fact that fish behaviour may be altered due to the presence of the observer. With any fish survey it must be borne in mind that fish are not static and tend to roam about within a general habitat area or territory. This makes the interpretation of any survey results problematical in terms of transferring the data directly to preference curves.

The size of the sample must also be sufficiently large. Bird (1996) quotes the work of Williamson in recommending a minimum sample size of 150.

5.3.3.2 'Transferability'

Due to economic constraints, many users are forced to use previously published data for species preference curves. Questions arise over the 'transferability' of data between watercourses. Species preference curves derived in one river may not be applicable on another. This may be due to the combined influence of the surrounding environment, or the influence of other factors which are not taken into account by PHABSIM. These include factors such as water temperature, the prevalence of predators, the density of fish populations, the amount of fish hiding places, and the supply of food. If any one of these factors is substantially different between two rivers, it is likely that the fish population may exhibit discernibly different behaviour. Gore and Nestler (1988) point to the work of Bovee and Gore which demonstrates superior results from the use of suitability curves derived on-site.

In addition, it is questionable whether preference curves that are derived from a watercourse where there is a preponderance of one of the types of hydraulic variables, can be used on another watercourse where there is only a small amount of that variable. For example, data collected on a river where the bed in almost completely composed of gravel should probably not be used to determine WUA on a river with only small areas of gravel bed. In this case, the physical environment of the two rivers will be too dissimilar to allow data to be transferred. Gore and Nestler (1988) refer to the work of Gore and Judy, and Morin et al who found that habitat suitability curves produced good predictions of fish density at stream sites with similar discharge patterns and amount of available habitat. If previously published preference curve information is to be used, it should be ensured that the data was obtained from a watercourse with as similar environment and range of hydraulic variables as possible. Bird (1996) quotes the work of Heggenes and Saltveit that states that the US Fish and Wildlife Service (USFWS) recommend that the 'transferability' of any preference curve information be validated before use on a different watercourse.

5.3.3.3 General Problems

Care needs to be taken over results that derive from the extreme limits of the preference curves. These areas of the preference function are often based on the observations of a small number of fish in a habitat that is of low availability. As a result, it is possible that actual preferences of the fish are misinterpreted due to the small probability ratios involved. Bird (1996) quotes the work of Williamson who recommends tolerance limits in the calculation of preference curves, whereby only frequencies that are above a set gauge level are used in calculations.

Another problem to note with the use of preference curves is that fish exhibit different habitat preferences in different seasons and between day and night. Bird (1996) refers to the work of Rimmer, Baltz and Maki-Petays to highlight this point. Despite this, it is common practice to use preference information based on surveys carried out during the daytime in summer, to predict WUA for the whole of the year.

An alternative to the use of species specific preference curves is the determination of community suitability information Gibbins et al (1996). In this method a number of sites are selected and the whole range of species found at each site are recorded, together with the hydraulic conditions at the time of sampling. The sites are sampled a number of times over a period to give a range of hydraulic conditions. Statistical analysis of the results enables the determination of a number of communities of species which are commonly found together. Suitability index values can be derived for the communities as a whole, as opposed to for individual species. However, unlike the usual IFIM approach, in this case the suitability index values take into account the availability of each of the types of localities sampled.

The method prescribed by Gibbins et al is presented in terms of invertebrates, however, the method would be equally applicable to fish. It overcomes some of the problems commonly associated with the use of suitability curves in the standard approach. Notably, communities are assessed instead of individual species. This approach has more relevance to river managers who cannot manage flow requirements based on the needs of single species, but must look at a wider approach. The method also does away with some of the problems of having to obtain suitability data for individual species. Difficulties occur in obtaining reliable data on separate species where abundance may be very low, or where other factors may be of great influence such as predation or chance events.

5.3.3.4 Selection of Discharge

Problems also occur with PHABSIM when attempting to test the correlation between the model predictions for WUA, and the biotic response of fish observed in the watercourse. The discharge in natural watercourses varies, and the question arises as to what discharge value should be used as input into the model. Common choices include mean monthly flow, median flow, lowest flow or some instantaneous value of flow. The choice is obviously significant, but the decision appears to lie at the discretion of the modeller.

5.3.3.5 Variability of Results

Based on the options that are chosen within PHABSIM, it is possible to take one set of hydraulic data, together with one set of species preference curves, and derive widely varying results for the amount of WUA Gan and McMahon (1990). Gan and McMahon (1990) also refer to the work of Milhous et al which demonstrated that the location of the critical detail in the discharge verses WUA relationship, i.e. the points of inflexion, can be affected by the choice of options. Gan and McMahon (1990) also highlight the work of Jensen who reported on studies carried out by three authorities using IFIM to produce discharge recommendations for the Stacy Dam on the Colorado River. The three different reports produced differing recommendations of 2.5-8 cusecs, 30-90 cusecs and 15-100 cusecs. This means that it is imperative that the user selects options based on biologically realistic assumptions, however, this is difficult as no biological consensus is commonly available.

5.3.3.6 Biological Interaction

IFIM gives no consideration to the numerous complex biological interactions that take place between aquatic species and their environment. This would include such factors as food supply, predation, competition and prey availability. However, Gore and Nestler (1988) refer to the work of Culp and Statzner to demonstrate that the influence of these factors is relatively small in comparison to the species interaction with the physical variables included in IFIM. In addition, to include biological interactions within the model in order to reflect conditions in the aquatic environment more accurately, would inevitably lead to increased complexity of the model.

5.3.3.7 Selection of Target Species and Life Stages

Bird (1996) makes some valid points about the use of species preference curves in IFIM. It is pointed out that to accurately model the habitat requirements of all of the species in a watercourse, and at each of the life stages, would be precluded by the amount of resources that would be necessary. As a result, 'target' species and life stages have to be selected, whilst others are not included in the modelling process. The selection of the target species and life stages obviously has great influence on the results obtained. As a species matures its requirements, in terms of the physical variables, alter and 'bottlenecks' can exist whereby a limit exists for the life stage most dependant on the habitat in short supply. It is suggested that it is most important to assess the available habitat for the younger life stages, rather than for the mature fish.

This is due to the fact that younger fish tend to remain in one area. As a result, they are affected to a greater degree by the prevailing conditions within the small region they inhabit, and are influenced far more by short term flow changes in the vicinity. It is also important to assess the available area for spawning, as this will be a limiting factor on the abundance of any species.

5.4. Studies utilising IFIM and/or PHABSIM

IFIM and PHABSIM have been used in a very wide range of habitat studies. In the first case considered here, PHABSIM was used to simulate available habitat for Atlantic Salmon in a small brook Bourgeois, Caissie and El-Jabi (1996). The conclusions derived from this thesis are fairly complimentary concerning PHABSIM's capabilities. The project revealed the robustness of the model to random measurement errors of up to 10% in water depth and 30% in velocities. These input errors produced a variation of only 4% in the WUA predicted by the model. Comparison between the results from PHABSIM and WUA measured on site, revealed an overall error of 7.5% in predicted values. In the majority of cases, PHABSIM was seen to be over predicting the available area. However, it was noted that the discrepancy between simulated and measured WUA was much lower at lower discharges. This is important as this is often the area of the WUA versus discharge curve that is of most importance. It was also found that the model was not sensitive to the type of substrate input to it. Finally, the authors tested for the models sensitivity to the choice of the location of transects. This produced increasing amounts of variability with increasing discharge. The amount of variability was found to be slightly higher than that found by introducing random errors into the input variables.

5.5. Discussion of IFIM and PHABSIM

Gore and Nestler (1988) suggest that PHABSIM be used as a comparative measure to assess the loss or gain in habitat from any set of proposals. However, since the results exhibit such a widely varying range, it is possible that selection of different assumptions will produce a completely different habitat loss or gain when considering alternate courses of action. As a result, great care needs to be taken in the derivation and interpretation of any results using these methods. The user needs to be fully aware of the limitations and assumptions implicit in the model. A fuller discussion of IFIM and the future of habitat modelling can be found in this thesis at the end of chapter 7.

5.6. Other Habitat Models

IFIM is the most widely used and discussed habitat modelling procedure, and forms the basis for a lot of the work undertaken in this thesis. However, a wide range of alternative approaches and models are available. It is only possible to pick out a few of these other methods for brief consideration here.

5.6.1 RCHARC

The Riverine Community Habitat Assessment and Restoration Concept (RCHARC) was developed by Nestler et al in 1993, at the U.S. Army Corps of Engineers Waterways Experimental Station Peters et al (1995). The approach seeks to take the main conceptual elements from IFIM, but simplify the approach substantially. The aim is to overcome one of the main problems in the application of PHABSIM, namely that the use of suitability curves from different species of fish, and a variety of life stages, results in a large amount of results that are difficult to analyse.

The twin fundamental concepts behind RCHARC are, firstly, that habitat quality and hydraulic and morphological diversity are strongly linked. Large variations in depth and velocity within a reach give rise to lots of good quality habitat. If there is little variability in the flow, for example in a channelised reach, the habitat quality will be poor. Secondly, that the principle of similarity between river reaches holds. Thus, the relationship between habitat quality and depths and velocities within one reach is the same as in another.

In order to implement RCHARC, distribution frequencies of depth and velocity of two reaches of river are compared. One reach being the one that has already been restored, and the other a reference or "comparison standard". The comparison standard river system (CSRS) is selected on the basis that it represents the target or ideal habitat conditions that the designer wishes to obtain in the restored reach. Usually each reach is subdivided into at least ten cross sections. Each cross section is further subdivided into 20 equally spaced cells. Thus, by measuring the depth and velocity in each cell, at least 200 velocity-depth recordings are taken per reach. These values are input to RCHARC which produces a three dimensional plot of velocity versus depth versus percentage occurrence for each of the reaches, at a particular discharge. By comparing the plots, a qualitative estimate is obtained of the similarity of the habitat between the two sites. This indicates the success in improving the physical habitat that has been delivered by the restoration scheme.

As well as using RCHARC to assess the success of restoration measures on a site, it can also be used in conjunction with a hydraulic modelling package, to choose between possible restoration alternatives. A hydraulic model can be used to predict velocities and depths following restoration, and these can be compared against existing values in the "comparison standard". The option which produces a frequency range of depths and velocities most closely approximating the standard would be the preferred choice, costs not withstanding.

The great benefit of RCHARC is that it is removed from the complex mosaic of problems, encountered in PHABSIM, which are associated with the use of species preference curves. Instead, the method deals only with the underlying hydraulic patterns of depths and velocities in two or more reaches, in order to assess habitat impacts on a community basis. The level of impact on the aquatic habitat, of any alterations to a watercourse, can be approximated by the changes in the frequency distribution of the physical variables relative to the target and standard reaches.

RCHARC has been tested on Rapid Creek, near Rapid City, South Dakota U.S.A. This river has been extensively restored following a massive flooding event in 1972, which resulted in the loss of two hundred and thirty five lives. The restoration included flood control measures together with habitat improvement features. The purpose was to prevent any further flooding, as well as to improve habitat for trout whose numbers had been substantially reduced by earlier channelization measures. A low flow channel was installed together with flow deflectors, pools, riffles and overhanging vegetation. The scheme was jointly designed by biologists, landscape architects and engineers.

RCHARC was used to assess the relative conditions between a restored reach of river, and a standard reach. Bivariate plots of velocities and depths in the two reaches were produced at a two discharges which represented a high and a low flow. The plots were found to display almost identical distributions. Field data further indicated that vegetative cover, dissolved oxygen levels and water temperatures were similar in the two reaches. As a result, it was reasonable to assume that the habitat would be similar in the sites in question, and therefore that the restoration had been a success. In fact, it was know that both reaches are now productive fisheries. This suggests that RCHARC is able to accurately predict the success of restoration schemes.

5.6.2 SERCON

The 'system for evaluating rivers for conservation' (SERCON) package provides a means of recording and analysing data from a river corridor survey. This type of survey is usually carried out before and after any scheme which may have an effect on the ecology of the river. A scoring system can be incorporated to assess the severity of any change. Data is input to SERCON on surveyed fish populations and the physical features of the river. The model then calculates a conservation rating for the site in question ranging from 0 for no value, up to 100 for highest value. The advantages of SERCON are that it provides a simple means of providing information to planners, it gives a representation of a site's restoration potential and it provides a useful measure of potential environmental impacts Johnson and Law (1995).

5.6.3 RIVPACS

An example of a package that is used as more of a predictive tool is the 'river invertebrate prediction and classification' (RIVPACS) developed by the Institute of Freshwater Ecology. This model is based upon a database of 8000 sites in England and Wales surveyed for physical and biological data. The 'biological monitoring working party' (BWMP) and 'average score per taxon' (ASPT) were computed at each location. These factors give a measure of species presence, abundance and diversity.

The methodology for use of RIVPACS is to assess the physical and chemical variables at any site under consideration, and then use the model to predict expected values for BWMP and ASPT. Any deficiency between numbers of invertebrates found from site sampling and predicted values from RIVPACS, indicate the presence of environmental stress at the location under consideration.

5.6.4 HABSCORE

A similar model to RIVPACS, but applicable to fish, is HABSCORE. Fish surveys together with a measurement of the prevailing physical variables at a number of sites, have been used to compile a database. The model is then able to make predictions of fish biomass at any new sites. Anomalies between these values, and populations determined from electrofishing, can be examined to determine the cause and decide what restorative measures may be possible.

The major cost of both RIVPACS and HABSCORE are in the development of the original database of information on sites. Application to specific sites is relatively cheap. The collection of the necessary data may only take one to two hours at any location.

5.6.5 CASIMIR

CASIMIR (computer aided simulation for instream flow requirements in diverted streams) was developed at the Institute of Hydraulic Engineering of the University of Stuttgart Giesecke and Jorde (1997). The purpose of developing the model was to investigate the impact of hydro power schemes on physical habitat. The approach used here is more complex and rigorous than in the PHABSIM method. The model considers the river in three parts: riparian zone, aquatic habitat and river bottom habitat.

The aquatic habitat is the body of water within the river and forms the habitat for fish. The model simulates this fish habitat in the same way as PHABSIM, by comparing predicted hydraulic variables with species preference curves. Bed shear stresses are calculated, and combined with information on the quality of the stratum, to simulate river bottom habitat for benthic organisms. Finally, survey detail on the floodplain is included in the hydraulic part of the model so that predictions of the periodic wetting and drying of the flood plain can be included in the results from the flow simulation. The predictions of temporary flooded areas are combined with known habitat potential for specific vegetation communities, to assess the effect upon habitat located on the flood plain.

5.6.6 Energetics models

In this approach, fish's habitat requirements are not reduced to a small number of preference curves, but instead fish physiology is considered Alfredsen (1998). Essentially in these types of model, the fish's foraging habits are considered. A model is created of the way in which a fish captures its food which uses a three dimensional flow model to compute the flow field in detail. The model calculates the maximum distance a fish will travel to capture food based on the fish velocity, stream velocity and prey velocity. The result of this is an energetics computation, which also takes into account water temperature, which can be used to determine the suitability of a given habitat. These types of model are still in their infancy and their potential use in habitat modelling is hard to evaluate.

5.7. Two Dimensional Habitat Modelling

Two dimensional habitat modelling is still a relatively new tool which has not yet been widely adopted by the modelling community. This is perhaps due to the unwillingness of end users to retrain to new technologies, and a fair degree of scepticism of the new programs on their part. Perhaps what is required, on the part of researchers, is more effort in biological validation of these 2D models to encourage more confidence in planners and managers.

Undoubtedly, 2D modelling holds some advantages over a 1D approach. In particular, 2D models are better able to describe the flow characteristics in complex situations i.e. in the vicinity of bends, meanders, braiding and islands. This is discussed in chapters 3 and 4. Other advantages and disadvantages are outlined below.

5.7.1 Spatially Explicit Habitat Models

It has already been stated that 2D models hold the advantage over their 1D counterparts in analysing the flow in complex situations. They are also better able to spatially describe the different habitat types that are available. Some researchers have suggested that it is the mosaic of habitat types available that determine the abundance of any particular species Bovee (1996). This implies that the importance of the prevalence of the specific range of values of the hydraulic variables for which the species in question exhibit the highest preference has been overstated. In fact, for optimum conditions fish require a variety of conditions within a relatively close proximity. Bovee (1996) points to the work of several authors who have found the importance of having different habitat conditions within a small area. For example, Freeman and Grossman pointed to the importance of eddy currents adjacent to areas of high velocity. Fausch and White and Southall and Hubert found that some salmonids were prevalent where velocity shelters were found close to high velocity currents. Rabeni and Jacobsen noted that smallmouth bass were to be found where mid channel pools were located adjacent to pools with structural cover. Kwak and Skelly reported that spawning redhorse suckers required pools adjacent to gravely riffles.

The actual amount of habitat available where two different types of conditions are available in close proximity, for example a deep pool adjacent to a riffle, can be quantified by the amount of contact, or edge, between the two areas. This contact zone can have a unique set of habitat characteristics. In order to quantify how much of this type of habitat is available requires a spatially explicit model. In this case a 2D model would be suitable, but a 1D model would be completely unable to describe the amount of interaction between the two adjoining zones.

As well as evaluating the amount of contact between habitat types, the spatially explicit nature of a 2D model allows the modeller to accurately distinguish habitat areas and their interaction. Bovee describes how Pringle et al and Townsend developed the idea of patch dynamics in habitat evaluation. Each patch represents a particular habitat type e.g. pool or riffle. The existence and interaction of the patches can only be determined by a spatially explicit model (i.e. not in 1D) and can be split into five factors (i) number of patch types, (ii) proportion of each type, (iii) spatial arrangement, (iv) patch shape, (v) contrast between neighbouring patches. Each of the five factors influences the available habitat in a unique way. The number of patches determines the degree of habitat diversity. The proportion of each type determines which is the predominant set of conditions. The spatial arrangement may affect the amount of foraging fish have to do for food and how dispersed a species is. Patch shape and size will determine the quantity and type of edge effects, small, spaced patches having larger edge effects than single big ones. Contrast between patches effects the type of edge effects between adjoining areas.

5.7.2 "Top-down" Habitat Suitability Criteria

The traditional approach to habitat modelling, as outlined in the sections on IFIM and PHABSIM, can be described as a "bottom-up" approach. Individual fish are sampled, and these observations are interpolated upwards to produce a histogram of habitat suitability ranging from 0 to 1. This is a successful approach where species exhibit well-defined habitat requirements. However, the approach is flawed where a species exhibits a variety of preferences for different habitat types in different conditions. The method also suffers where it is the interaction of habitat types that are of importance to the fish, or the fish are too rare or widely spaced to observe.

Spatially explicit models offer the possibility of combining a habitat mosaic with information on species abundance. This gives rise to the possibility of a "top-down" strategy where an entire reach is assessed at one time to evaluate the determining or important factors in community structure and function. Using powerful statistical techniques it is possible to investigate the affects of the determining variables. Traditionally, the determining factors are taken as depth, velocity and substrate. However, this method allows the development of "synthetic variables" on which to base the spatially explicit indexes. These indexes may indicate relationships between the modelled habitat mosaic and the observed fish species that would not be evident using the traditional one variable at a time approach. A full discussion of the statistical techniques and methodology relevant to evaluating a 2 dimensional habitat model is beyond the scope of this text.

A "top-down" approach to suitability criteria is based at a larger scale (termed mesohabitat scale) than the traditional approach. In the IFIM/PHABSIM procedure, high precision sampling is a fundamental requirement. The difficulties in obtaining accurate survey information have already been outlined in the section on IFIM and PHABSIM. In the "top-down" approach, accurate surveys are less essential as attention is focused at a much wider scale. It is the mesohabitat conditions, the conditions in the area as a whole not at the point a specific fish is observed, in comparison with the observed characteristics of entire fish communities that is of significance. This overcomes a major weakness of the "bottom-up" procedure.

5.7.3 Data Input and Model Verification

In general, 2D models are more flexible to the data that can be input, whereas 1D models require data to be in the form of cross sections which are transverse to the stream banks. The modeller can use more discretion in determining what data to collect and input to a 2D model, and utilise more points that are at locations that are more interesting from a biological point of view. In a study on the Waterton River, Alberta, Canada it was found that a 2D finite element model required less bed level data to accurately simulate velocities in the river Ghanem et al (1996). In addition, the 1D model required a good deal of velocity measurements for verification. This is due to the fact that the calculation of the velocity distribution across the channel in the 1D model is not based on a physically sound hydraulic simulation, but on regression analysis and interpolation. The 2D model, which solves a second momentum equation to determine the velocities across the channel width, required just a few velocity measurements for verification.

As has already been stated, 1D hydraulic models require extensive velocity measurements to be taken in order to accurately validate the model. This is due to the fact that the model is not based on physically realistic assumptions when determining the velocities across the channel width. As this is the case, 1D models are essentially limited to predictions within the observed range of discharges used for model calibration. Extrapolation to discharges outside of this range is likely to result in errors. In addition, the model is unlikely to be able to accurately predict velocities resulting from proposed alterations to the channel, for example by some form of restoration or rehabilitation. In this case, a two dimensional model is likely to be much more reliable in predicting post scheme velocities Boudreau et al (1996).

5.7.4 Comparison of Results from 1D and 2D Habitat Modelling

In order to evaluate whether the additional expense of carrying out 2D habitat modelling over a 1D approach is worthwhile, a number of authors have carried out comparisons between predicted results from a variety of models. In the first case considered here, IFIM was used to provide a comparison between WUA predictions produced by 1D and 2D models on the Logan River, Utah, USA Tarbert and Hardy (1996). The authors used RMA-2 as the 2D model, and carried out their own calculations to approximate the hydraulic results produced from PHABSIM for the 1D case. The study revealed that the 1D model was able to obtain good agreement with RMA-2 for the simulation of the hydraulic variables. This was dependent on the fact that the cross sections used in the 1D model were sufficiently close together (24 metres apart in this case). The study calculated WUA for adult brown trout. There was very little difference in the amount of habitat calculated with the 1D and 2D approaches.

The authors suggest that their results indicate that the differences between a 1D and 2D approach to habitat evaluation become very small provided that their is sufficient spatial detail in the 1D model. However, this may be an over simplification. It is possible that two dimensional effects were negligible in this case. In other cases, however, the flow structure may be such that two dimensional effects are of considerably greater importance.

In another study, on the Atlantic Salmon population of the Moisie River (Quebec), a two dimensional finite element model was used to compute depths and velocities in the reach under consideration Leclerc et al (1995). Two sites were considered and a wide range of discharges were simulated. The model was able to predict velocities to an accuracy of 10% and discharge to an accuracy of 2%. At each node in the model, the predicted hydraulic variables were compared with habitat suitability information which was derived from fish sampling. This allowed for detailed mapping of the spatial distribution of the available habitat in terms of the suitability index values. In addition, WUA for each site was calculated by integrating the index values with cell areas. The study revealed that the resident Atlantic Salmon were insensitive to the projected range of flows simulated. The authors concluded that the improved accuracy and resolution of the 2D model contributed to a better understanding of the likely effects of altering the physical habitat variables. Increased research into using IFIM combined with a 2D approach would improve understanding of many of the shortfalls that are related to the biological aspects of the methodology.

5.8. Three Dimensional Habitat Modelling

Three dimensional habitat modelling offers a further improvement over a two dimensional approach, allowing more accurate calculation of flow velocities. Few studies have been carried out using three dimensional flow models for habitat assessment. This is perhaps due to the fact that 3D simulations can be very demanding in terms of computing power and data storage. There is also a lack of a validated 3D modelling package which can easily be used to model natural watercourses. One case of the application of three dimensional habitat modelling is on the Gjengedal river in western Norway Olsen and Alfredsen (1996). Here the authors present their own model, SSIIM, for calculation of the flow field. The software solves the Navier-Stokes equations with the k-ε model on a non orthogonal grid. The position of the free surface is first determined using the one dimensional package HEC-2.

It is suggested that one of the advantages of using a 3D model is that it is not necessary to calibrate the models predicted velocities against measurements taken on site. It is presumed that this is due to the expected accuracy of the 3D approach. However, close checking of the predicted velocities may be considerably more essential than the authors suggest, as the application of 3D models to predicting velocities in natural watercourses is still a relatively new field. In particular, problems exist over the accurate representation of wall roughness. This plays a vital part in the determination of the velocity field. Some discussion of this problem is given in chapter 3.

One advantage of a three dimensional approach is that it is possible to utilise the velocities predicted by the model to determine the bed shear stresses and hence combine habitat predictions with predictions of likely areas of erosion and deposition. This can give a complete picture of the affect of any engineering scheme on the river in terms of habitat changes and morphological effects.

5.9. Habitat Modelling of the River Idle Rehabilitation Scheme

Two FORTRAN programs were written from scratch, for the purposes of this thesis, and used to assess the improvement in habitat arising out of the rehabilitation of the River Idle. The programs, and the results that were produced using them, are detailed below. Some discussion of the results obtained from the programs, and the limitations in the approach used are given at the end of the chapter.

5.10. Habitat Programs

Two programs, COMBSIM1D and COMBSIM3D, were written by the author in order to apply the IFIM methodology (outlined earlier in the chapter) to the results from ISIS and CFX respectively. It was the initial intention to also write a program to apply the IFIM methodology to the results from SSIIM. However, as is detailed in chapter 4, the results from SSIIM calibrate quite poorly with site measurements (both in magnitude and in terms of the predicted flow structure) so it was decided to omit any habitat analysis of the two dimensional results.

Each version of COMBSIM takes predicted values of depth, velocity and substrate and compares them with preference curves for a number of target species. The preference curve information was taken from the Institute of Hydrology Report No. 115 Bullock et al (1991). The target species for the thesis were chosen to include both game and coarse fish, and different life stages. The species selected were adult trout, juvenile trout, adult chub, spawning chub and roach fry.

The work outlined here focuses on available habitat for fish only. This method was used because it is the most common approach to habitat modelling. There may be some deficiencies in excluding microinvertebrates from the analysis and this was discussed in the earlier literature review.

It is believed that it is the application of the techniques that are used here that is of more significance than the final predictions of weighted usable area. It should also be borne in mind that predictions of weighted usable area represent the amount of habitat that is available, and not a quantity of fish biomass.

5.10.1 COMBSIM3D

COMBSIM3D takes the results from the three dimensional modelling program CFX and computes habitat suitability indices and weighted usable areas for the five target species. In order to enable CFX to generate the ASCII data required for subsequent analysis by COMBSIM3D, it was necessary to write a short piece of user FORTRAN which is used as part of the input for a simulation run. CFX contains FORTRAN subroutines which the user can modify. These pieces of FORTRAN have a wide range of applications, but in this case the purpose was to instruct the program to output velocity and co-ordinate information, for each flow cell, to a separate file at the completion of the final iteration of the simulation run.

COMBSIM3D begins by reading in the ASCII information generated by CFX. It then calculates a resultant velocity for each flow cell based on the longitudinal and transverse velocities in that cell. The vertical velocity is ignored because its contribution to species habitat is not known. Velocity related suitability indices are calculated, for each species, by comparing the resultant velocities, for each cell, with the information contained in the species preference curves.

The next stage is to calculate a substrate related suitability index value. Unfortunately, the substrate surveys that have been taken at the deflector sites, both pre and post installation, are not at sufficiently close spatial intervals to provide meaningful input to the program. However, surveys following the deflector installations have shown that the sand bed is eroded adjacent to the tip of each of the deflectors to form a pool with a clean gravel substrate. As a result, it was necessary to devise some means of predicting whether the bed was composed of sand or gravel at each point, and hence allocate the appropriate suitability index.

In order to do this for each of the discharges studied, the flow velocities adjacent to the bed were plotted in each case. A reference velocity was chosen such that it was not exceeded prior to the installation of the deflectors (where the bed was known to be wholly composed of sand). The reference velocity was exceeded in the post deflector simulation, in an area of river adjacent to the deflector tip which roughly matched the position where the pool was known to develop in reality. In the area where the resultant velocity exceeded the reference velocity, COMBSIM3D assigned a suitability index (for each target species) in accordance with a gravel substrate. In the remainder of the domain, a suitability index in accordance with a sand bed was used. The theory behind this is that the faster the water is moving near the bed the higher will be the bed shear stress, and hence the greater is the probability of the sand substrate being eroded.

An alternative method for determining the areas of the bed that would be composed of sand, and areas of gravel, would be to use a different type of subroutine based around the methodology of the SHEAR program discussed in chapter 6. This would involve calculating the bed shear stress and comparing it to a critical value for the erosion of the bed sediment. This approach would involve certain problems, and these are discussed in the future work section of the conclusions chapter.

Next the program calculates the depth averaged velocity at each node point, throughout the solution domain. These depth averaged velocities are used to assign a second suitability index value, at each node point, and for each of the target species.

For the third, depth related, suitability index it is necessary to calculate the depth at each of the node points where the velocities are known. The set of depths for the solution domain are input separately to COMBSIM3D from a text file. At each of the nodes where the velocity is known, COMBSIM3D calculates a weighted average depth based on the two nearest values. A depth related suitability index value, for each of the target species, is then calculated at the same node points as the previously calculated depth and substrate suitability indices. A combined suitability index value, for each target species and at each node, is calculated by multiplying the three suitability index values together. A weighted usable area for each target species can be determined by multiplying the plan area of each of the flow cells by the combined suitability index value at that point, and summing the result for the whole of the solution domain.

5.10.2 COMBSIM1D

COMBSIM1D works in very similar fashion to COMBSIM3D, except that it computes habitat information based on the results from the one dimensional hydraulic modelling program ISIS.

Despite the fact that ISIS, being a one dimensional modelling program, normally computes only one velocity per cross section, it can be used to calculate a number of velocities across each section. ISIS refers to these as panel velocities. ISIS calculates panel velocities by determining the conveyance for each panel and adjusting them by the total conveyance of the cross section, to calculate an average velocity for each part of the cross section. More detail on ISIS, panel velocities and one dimensional modelling in general is given in chapter 3. The panel velocities are used by COMBSIM1D to calculate velocity related habitat suitability indices.

Panel velocities are a useful way of generating velocities across the width of each of the cross sections. They provide information on the flow pattern such that a spatial assessment of the available habitat can be attempted. Thus, shaded colour plots of habitat indices can be produced for the 1D as well as the 3D analysis. However, the panel velocities should at best be considered as an approximation of the likely flow pattern. They are not produced by solving the full momentum equations in each of the three mutually perpendicular directions, in the way that the three dimensional results are, and so cannot be considered to be reliable.

The velocities generated by the one dimensional modelling part of the PHABSIM software (discussed earlier in the chapter) have to be carefully calibrated against site measurements of velocity to ensure they are actually representative of the real situation. Calibration in this sense essentially appears to mean altering the model predicted values to what has actually been recorded on site. The panel velocities produced using ISIS have not been calibrated as the operation would have been too lengthy and, in any case, it was felt that it may be of more interest to directly compare 1D and 3D predictions.

5.11. Results

Results are presented, for the three deflector sites on the River Idle, in terms of colour coded, shaded contour plots of the combined suitability indices for each of the target species, and at a number of different discharges figures 5.3 to 5.30. These plots were prepared using the Gsharp data visualization tool developed by Advanced Visual Systems. It is not possible to include the results from COMBSIM3D for every target species at every discharge. Instead, a number of key cases have been selected that highlight the main themes evident in the results. A lesser number of results plots from COMBSIM1D are also included to highlight the results that can be obtained using the different approaches. Graphs of weighted usable area of habitat for pre and post scheme are also given, figures 5.31 to 5.43. Weighted usable area graphs based on the results from COMBSIM1D are only included at the initial discharge. This provides sufficient data for comparison with 1D whilst limiting the number of plots that are included.

The graphs showing available areas of habitat indicate that there is virtually no available habitat for adult trout at the lower flows simulated. As a result, it is to be expected that the Idle will not be colonised by trout. Consequently colour coded, shaded contour plots are not given for either of the life stages of trout that were simulated using COMBSIM3D and COMBSIM1D. Instead, colour habitat plots are presented for the three other life stages of target species i.e. adult chub, spawning chub and roach fry. Chub and roach are very common coarse fish that would typically be found in a river such as the Idle.

Combined Habitat Suitability Index
Spawning Chub
Deflector 3c - Initial Flow
Three Dimensional Results

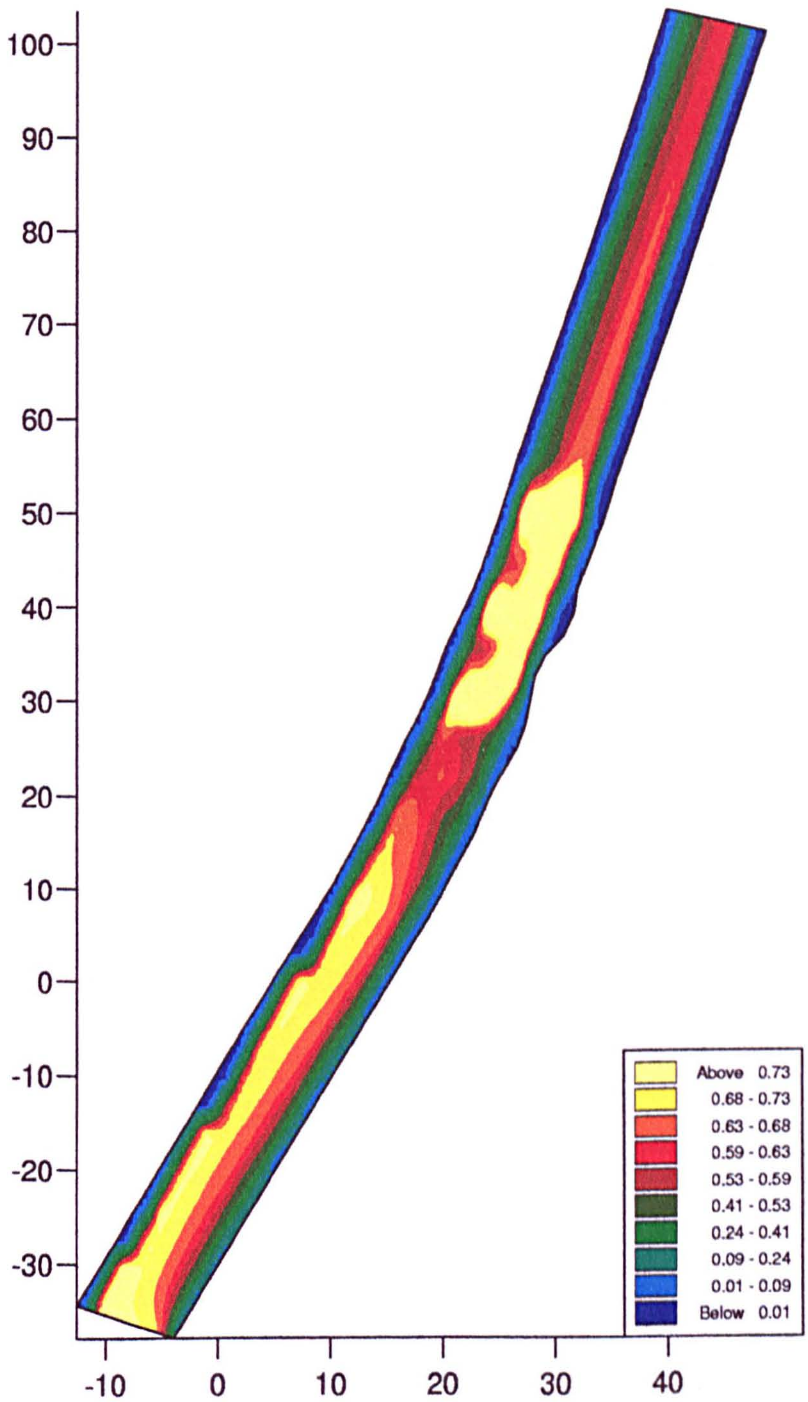
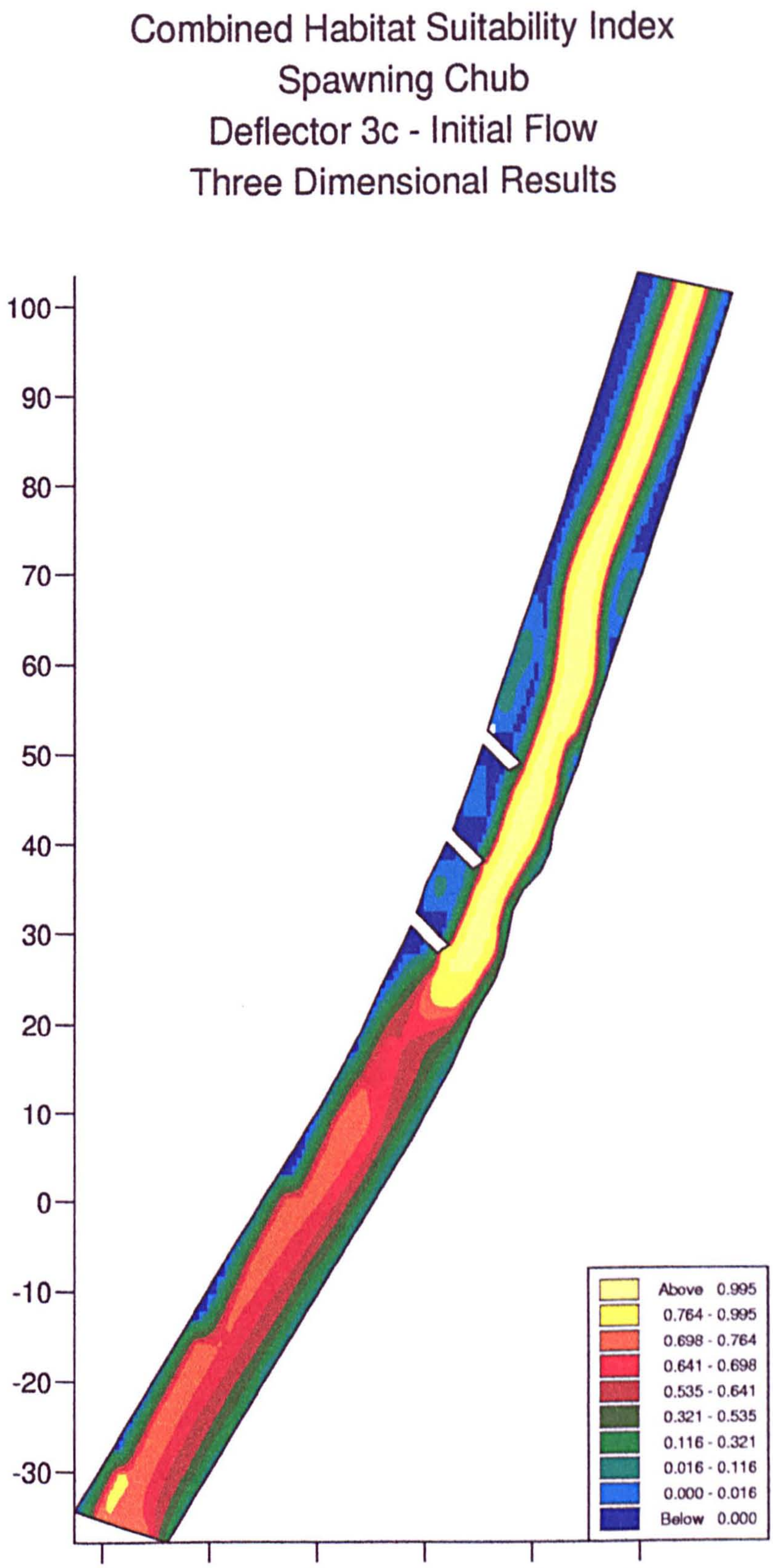
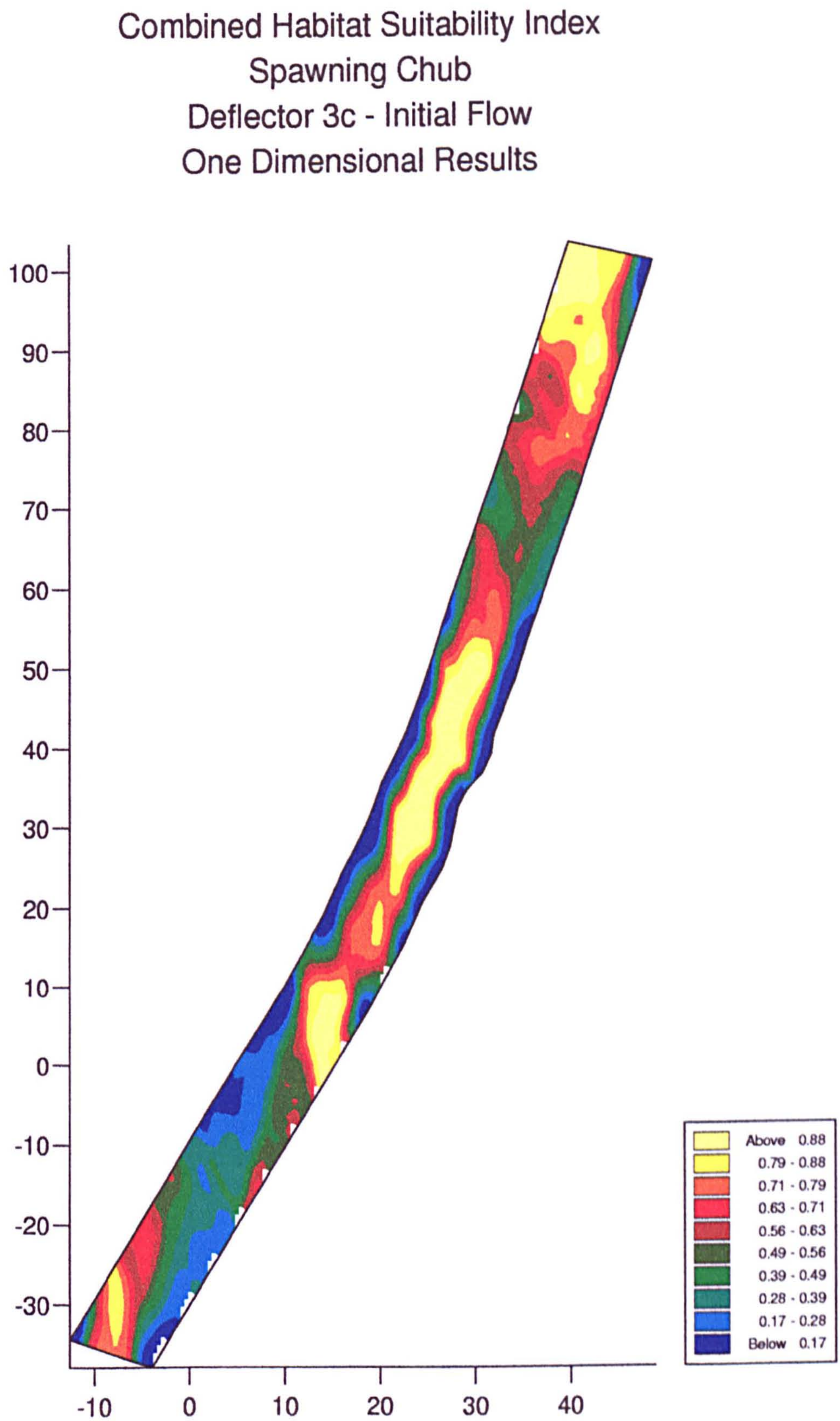


Figure 5.3:



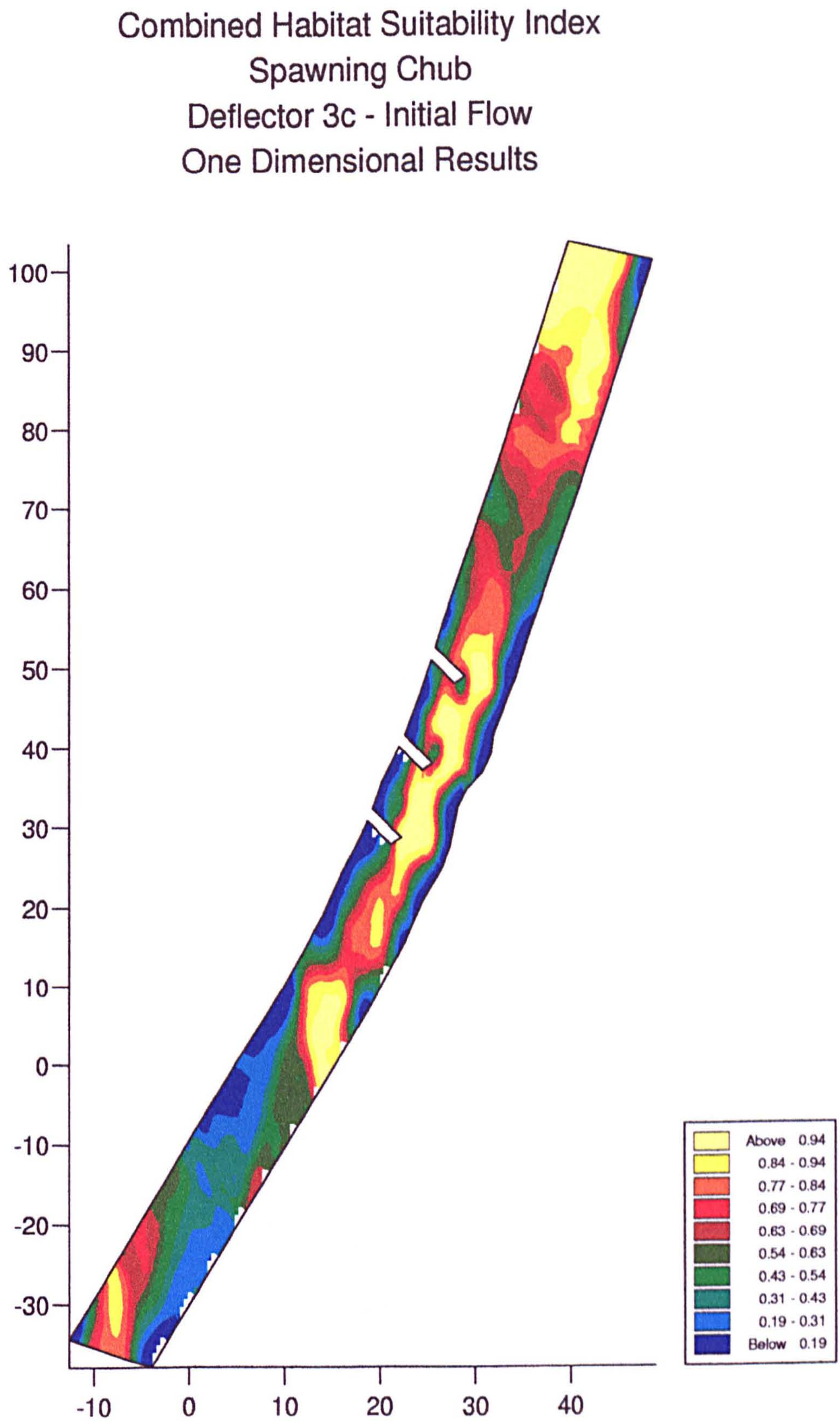
Combined Habitat Suitability Index
Spawning Chub
Deflector 3c - Initial Flow
Three Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.995
0.764 - 0.995
0.698 - 0.764
0.641 - 0.698
0.535 - 0.641
0.321 - 0.535
0.116 - 0.321
0.016 - 0.116
0.000 - 0.016
Below 0.000

Figure 5.4:



Combined Habitat Suitability Index
Spawning Chub
Deflector 3c - Initial Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.88
0.79 - 0.88
0.71 - 0.79
0.63 - 0.71
0.56 - 0.63
0.49 - 0.56
0.39 - 0.49
0.28 - 0.39
0.17 - 0.28
Below 0.17

Figure 5.5:



Combined Habitat Suitability Index
Spawning Chub
Deflector 3c - Initial Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.94
0.84 - 0.94
0.77 - 0.84
0.69 - 0.77
0.63 - 0.69
0.54 - 0.63
0.43 - 0.54
0.31 - 0.43
0.19 - 0.31
Below 0.19

Figure 5.6:

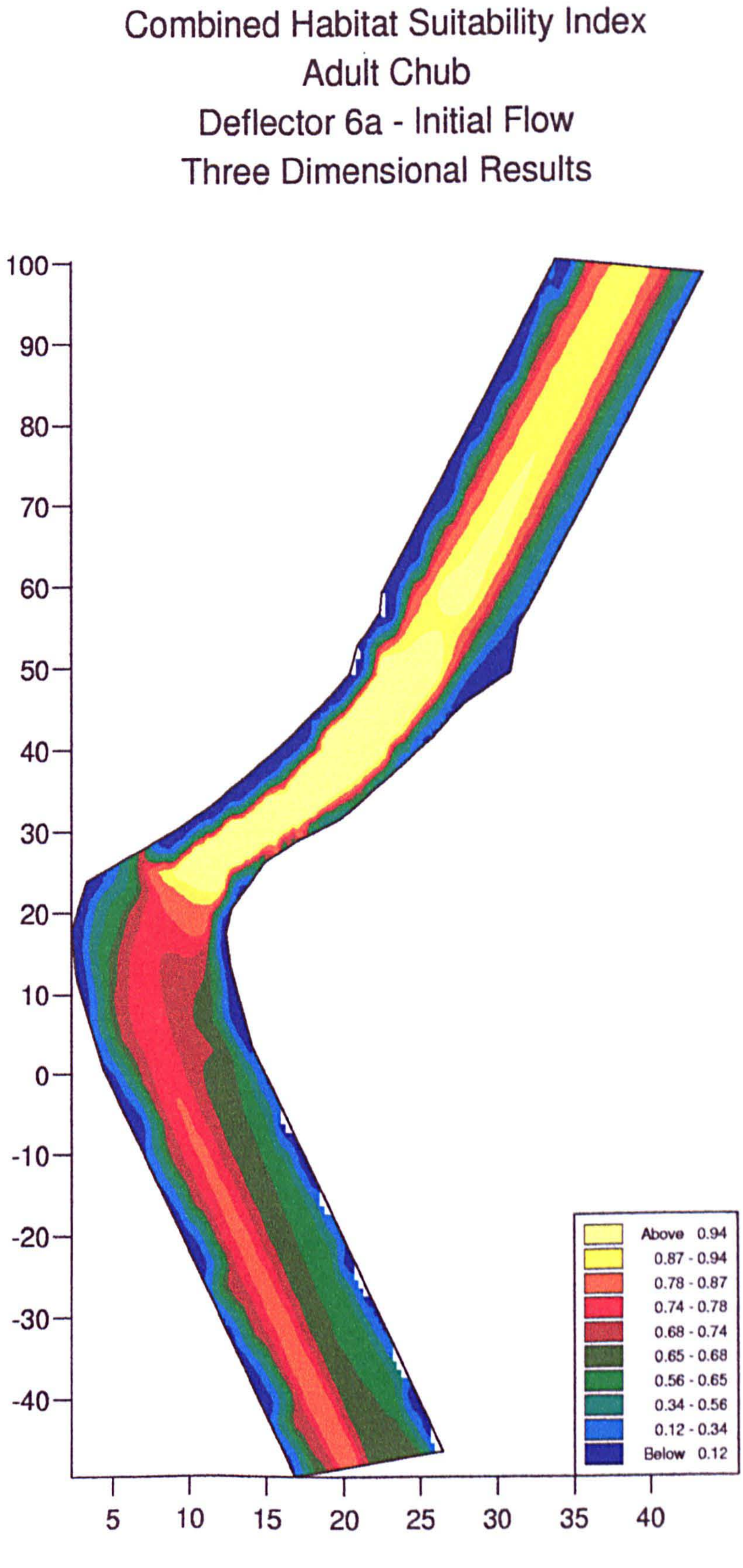

Combined Habitat Suitability Index
Adult Chub
Deflector 6a - Initial Flow
Three Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
5
10
15
20
25
30
35
40
Above 0.94
0.87 - 0.94
0.78 - 0.87
0.74 - 0.78
0.68 - 0.74
0.65 - 0.68
0.56 - 0.65
0.34 - 0.56
0.12 - 0.34
Below 0.12

Figure 5.7:

Combined Habitat Suitability Index
Adult Chub
Deflector 6a - Initial Flow
Three Dimensional Results

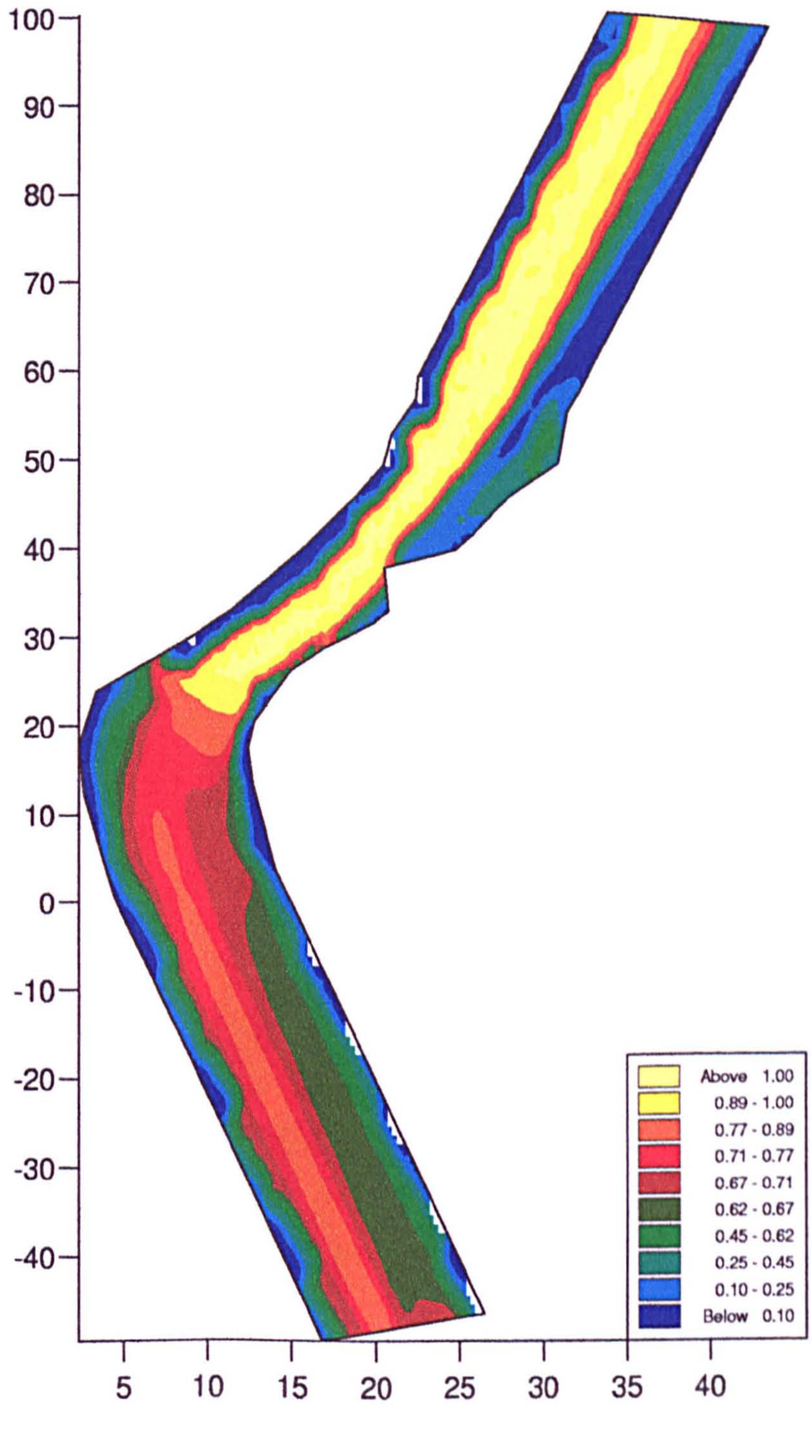


Figure 5.8:

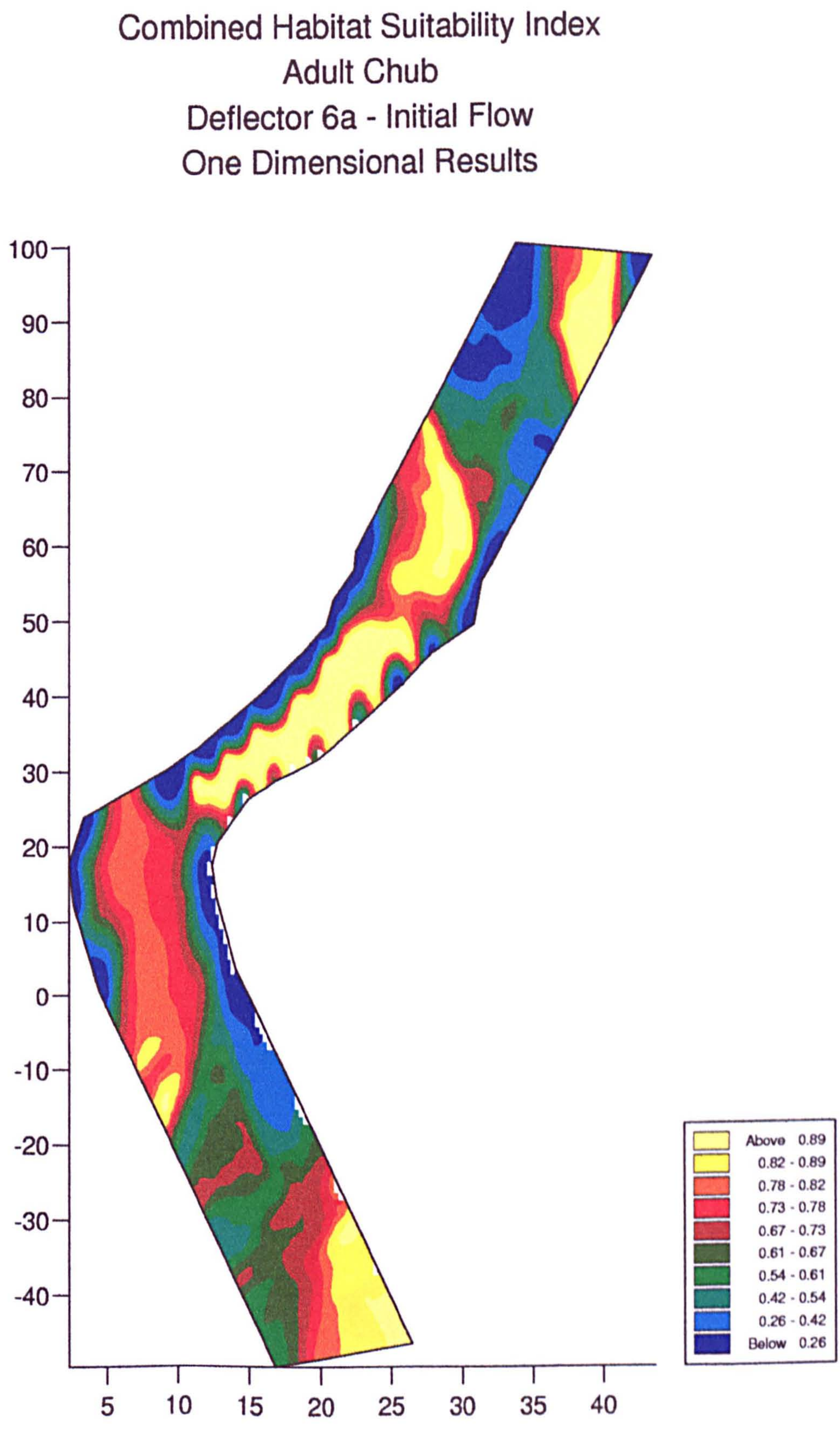

Combined Habitat Suitability Index
Adult Chub
Deflector 6a - Initial Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
5
10
15
20
25
30
35
40
Above 0.89
0.82 - 0.89
0.78 - 0.82
0.73 - 0.78
0.67 - 0.73
0.61 - 0.67
0.54 - 0.61
0.42 - 0.54
0.26 - 0.42
Below 0.26

Figure 5.9:

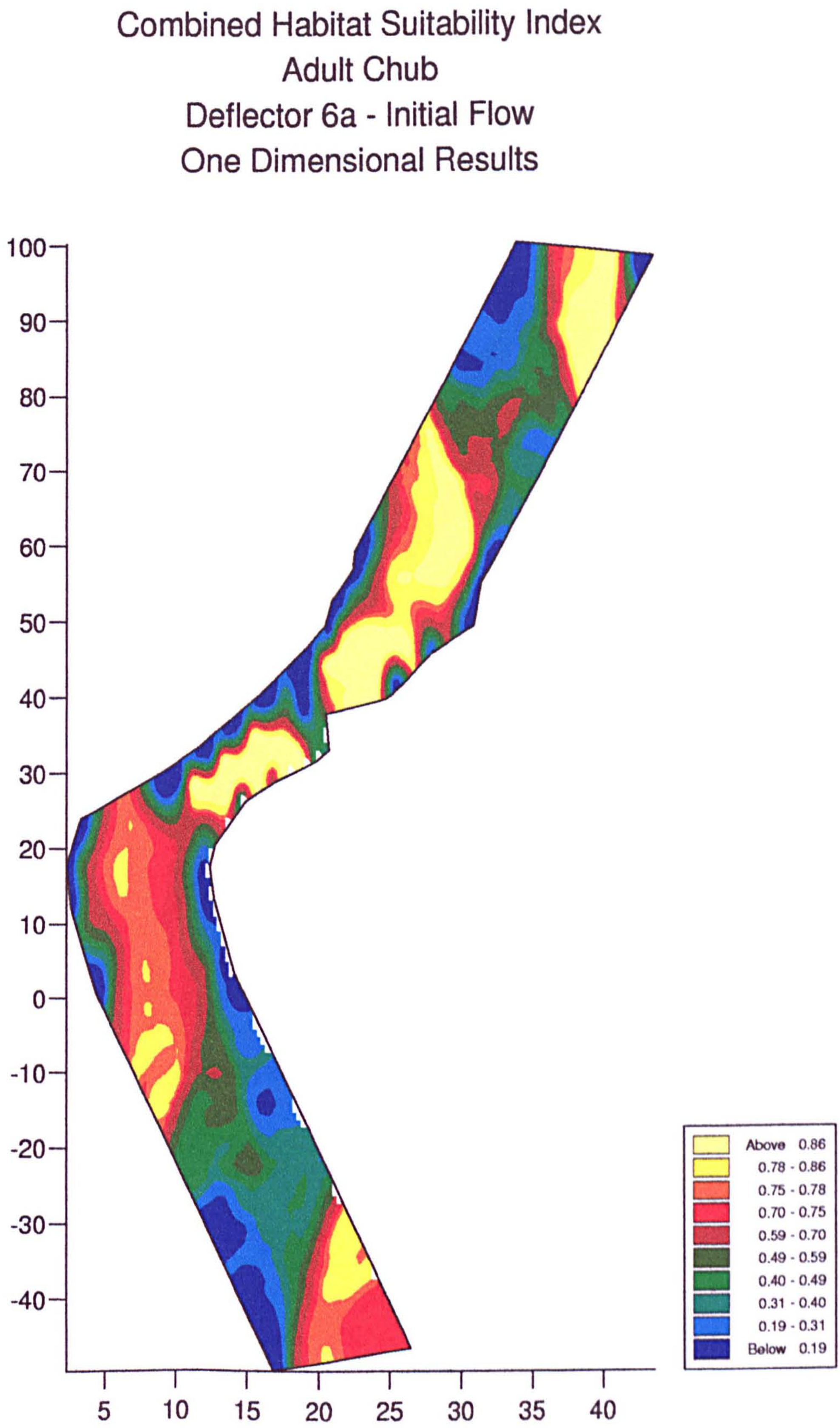

Combined Habitat Suitability Index
Adult Chub
Deflector 6a - Initial Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
5
10
15
20
25
30
35
40
Above 0.86
0.78 - 0.86
0.75 - 0.78
0.70 - 0.75
0.59 - 0.70
0.49 - 0.59
0.40 - 0.49
0.31 - 0.40
0.19 - 0.31
Below 0.19

Figure 5.10:

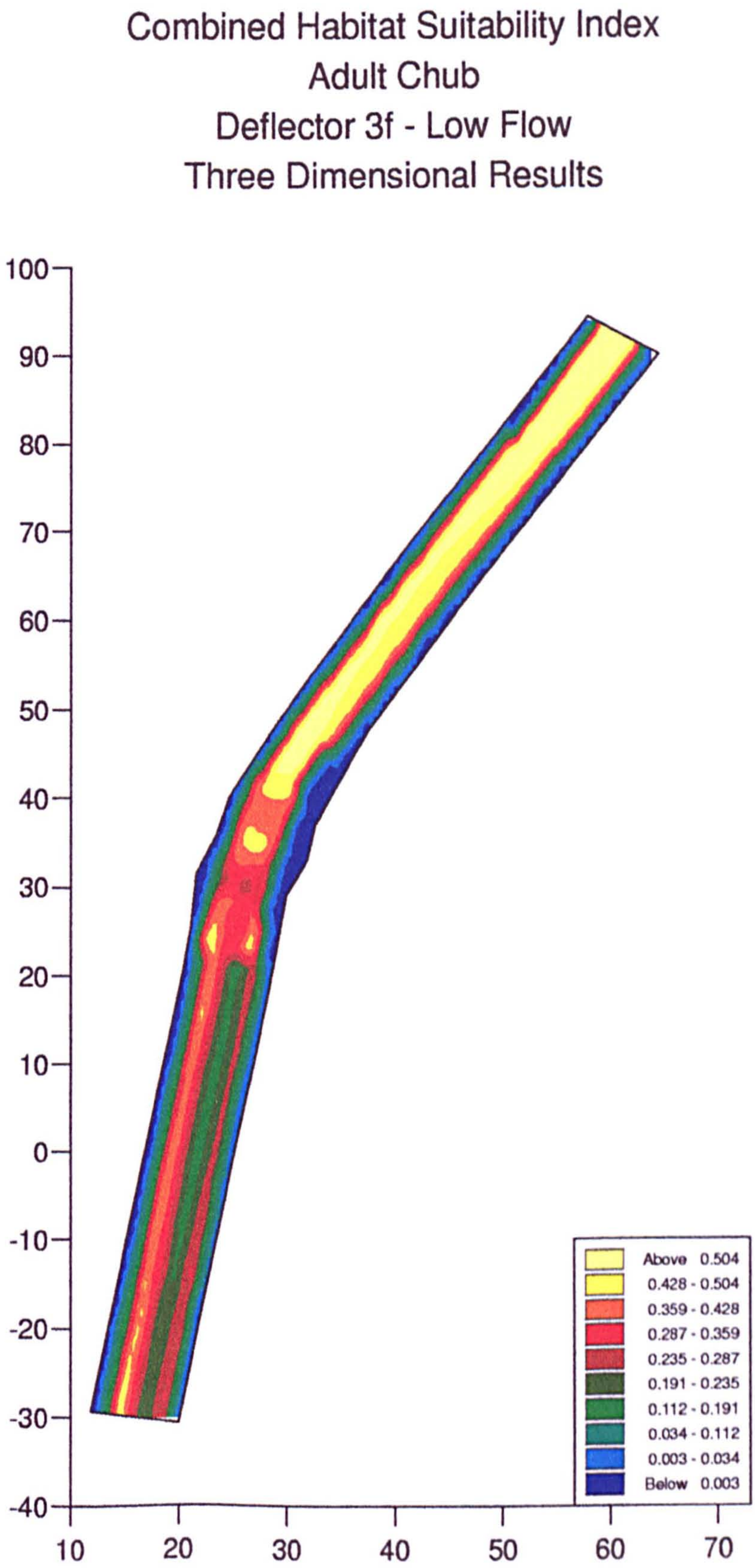

Combined Habitat Suitability Index
Adult Chub
Deflector 3f - Low Flow
Three Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.504
0.428 - 0.504
0.359 - 0.428
0.287 - 0.359
0.235 - 0.287
0.191 - 0.235
0.112 - 0.191
0.034 - 0.112
0.003 - 0.034
Below 0.003

Figure 5.11:

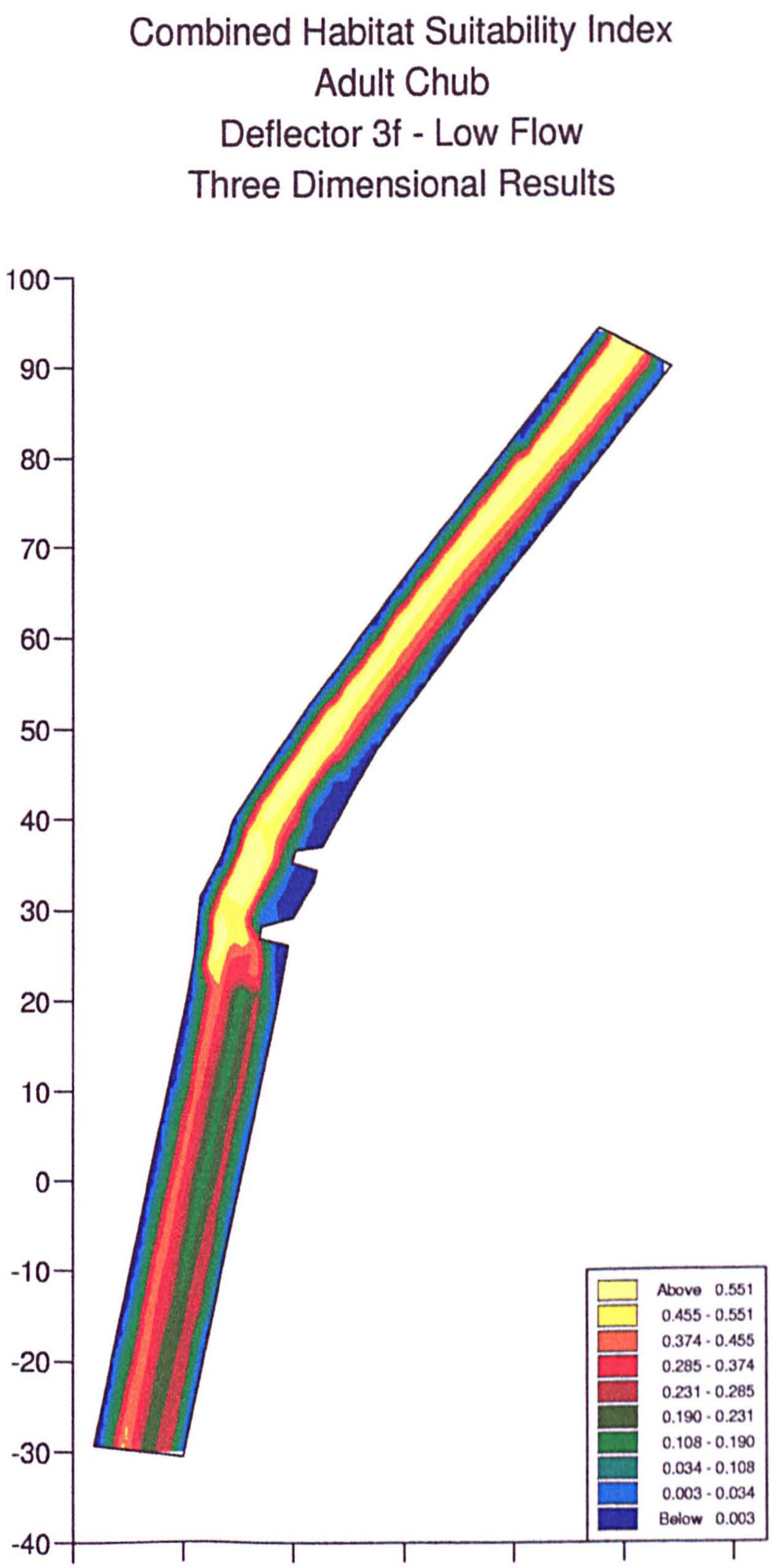

Combined Habitat Suitability Index
Adult Chub
Deflector 3f - Low Flow
Three Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.551
0.455 - 0.551
0.374 - 0.455
0.285 - 0.374
0.231 - 0.285
0.190 - 0.231
0.108 - 0.190
0.034 - 0.108
0.003 - 0.034
Below 0.003

Figure 5.12:

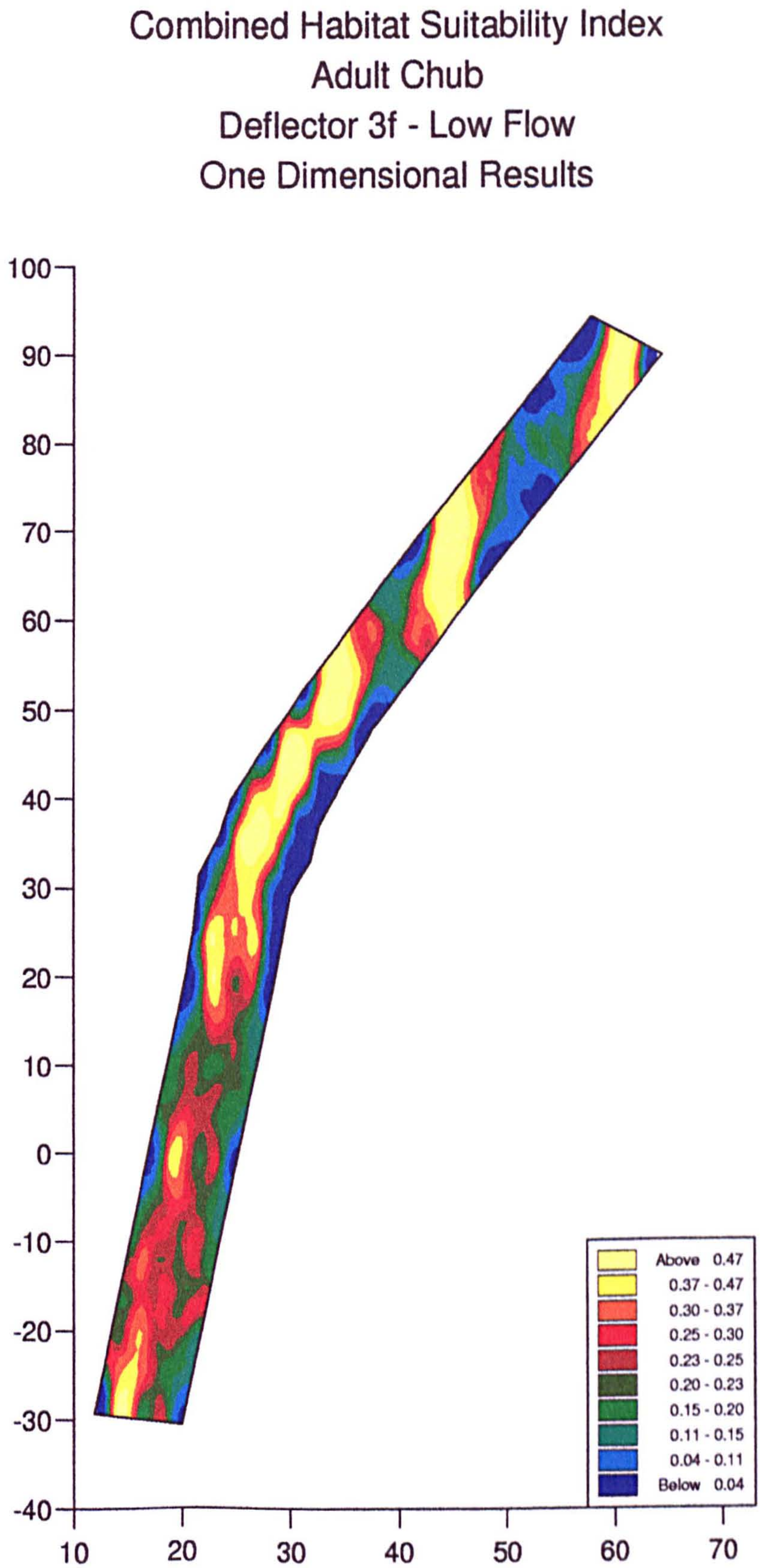

Combined Habitat Suitability Index
Adult Chub
Deflector 3f - Low Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.47
0.37 - 0.47
0.30 - 0.37
0.25 - 0.30
0.23 - 0.25
0.20 - 0.23
0.15 - 0.20
0.11 - 0.15
0.04 - 0.11
Below 0.04

Figure 5.13:

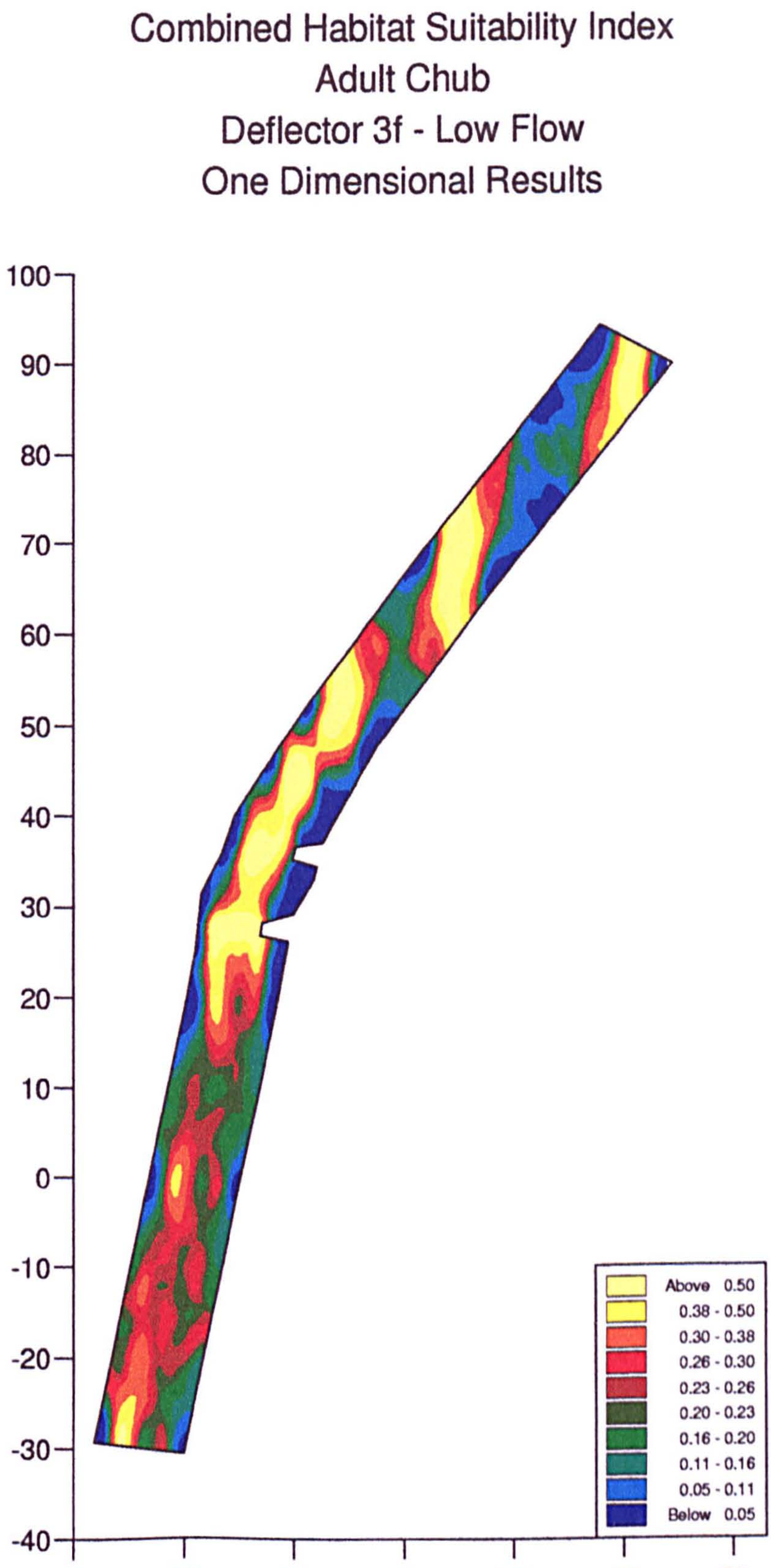

Combined Habitat Suitability Index
Adult Chub
Deflector 3f - Low Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.50
0.38 - 0.50
0.30 - 0.38
0.26 - 0.30
0.23 - 0.26
0.20 - 0.23
0.16 - 0.20
0.11 - 0.16
0.05 - 0.11
Below 0.05

Figure 5.14:

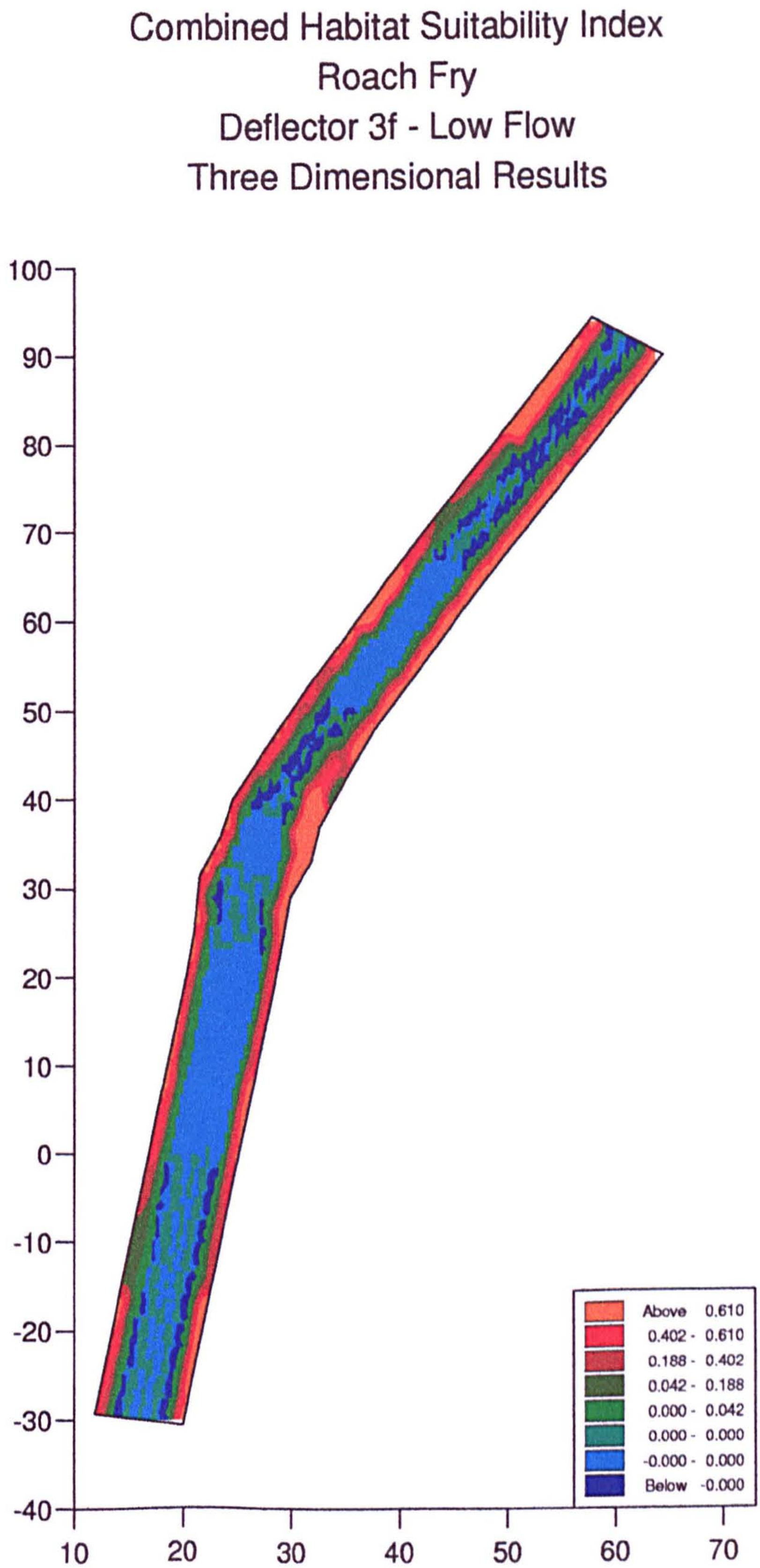

Combined Habitat Suitability Index
Roach Fry
Deflector 3f - Low Flow
Three Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.610
0.402 - 0.610
0.188 - 0.402
0.042 - 0.188
0.000 - 0.042
0.000 - 0.000
-0.000 - 0.000
Below -0.000

Figure 5.15:

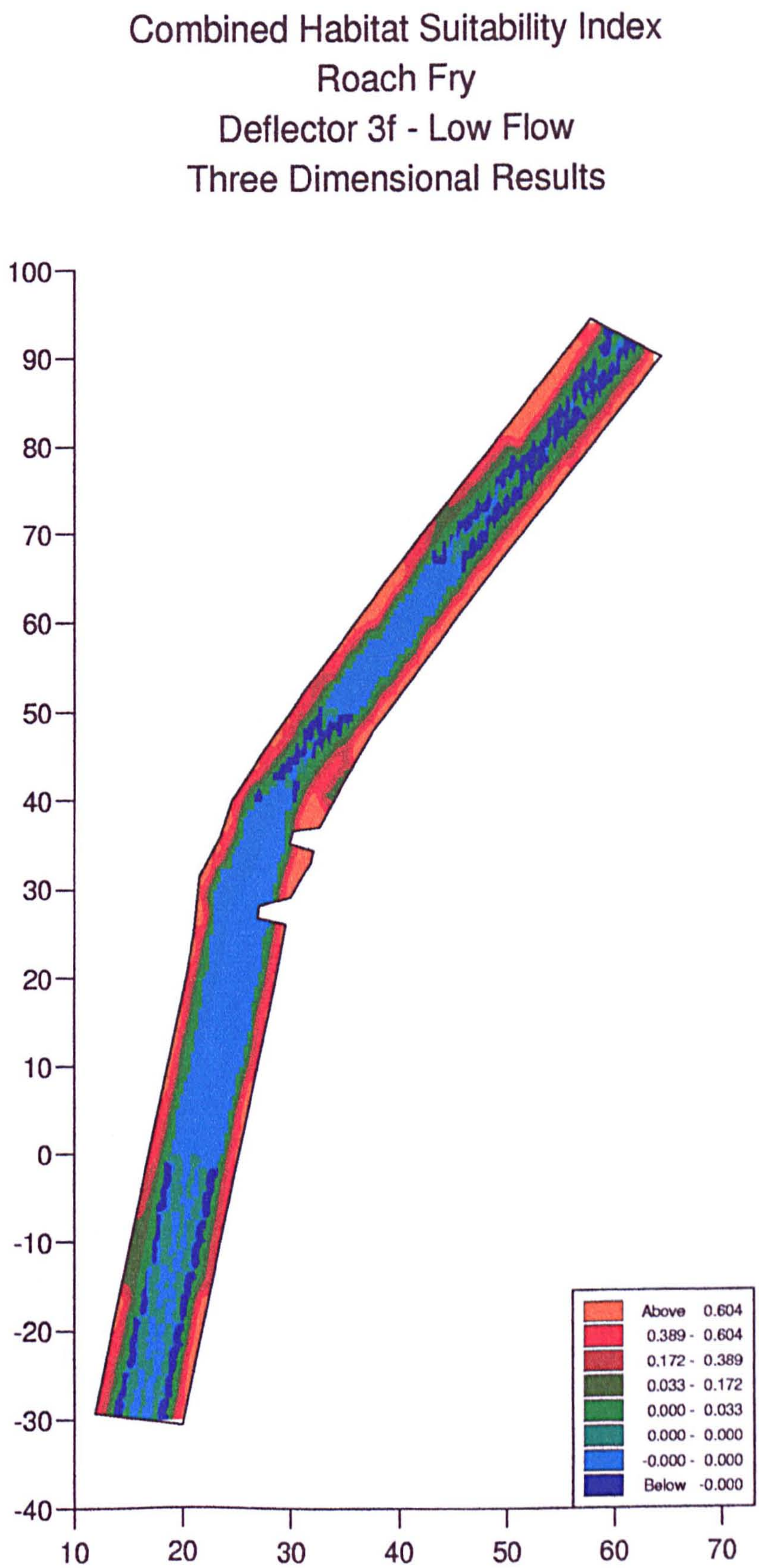

Combined Habitat Suitability Index
Roach Fry
Deflector 3f - Low Flow
Three Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.604
0.389 - 0.604
0.172 - 0.389
0.033 - 0.172
0.000 - 0.033
0.000 - 0.000
-0.000 - 0.000
Below -0.000

Figure 5.16:

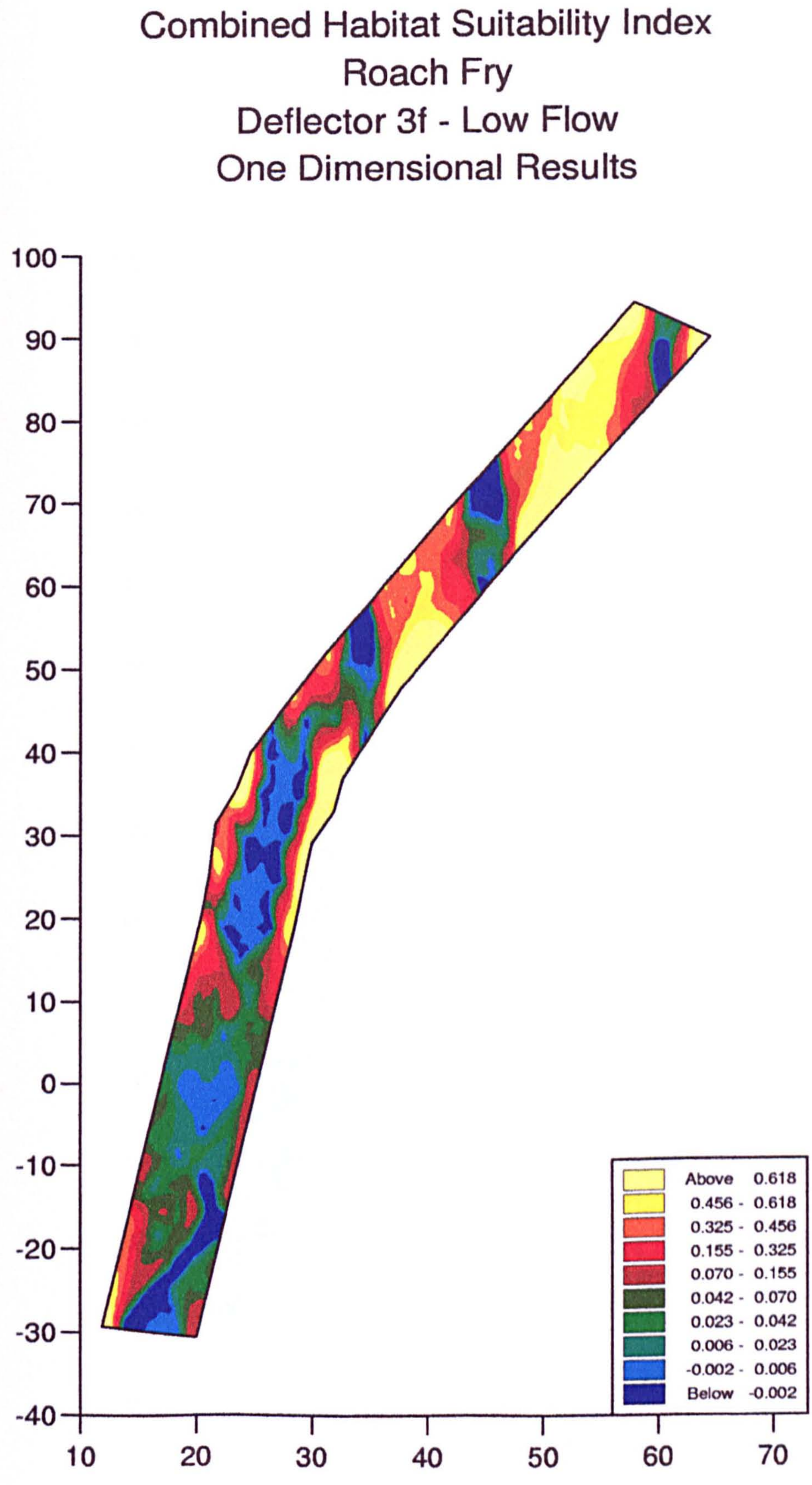

Combined Habitat Suitability Index
Roach Fry
Deflector 3f - Low Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.618
0.456 - 0.618
0.325 - 0.456
0.155 - 0.325
0.070 - 0.155
0.042 - 0.070
0.023 - 0.042
0.006 - 0.023
-0.002 - 0.006
Below -0.002

Figure 5.17:

Combined Habitat Suitability Index
Roach Fry
Deflector 3f - Low Flow
One Dimensional Results

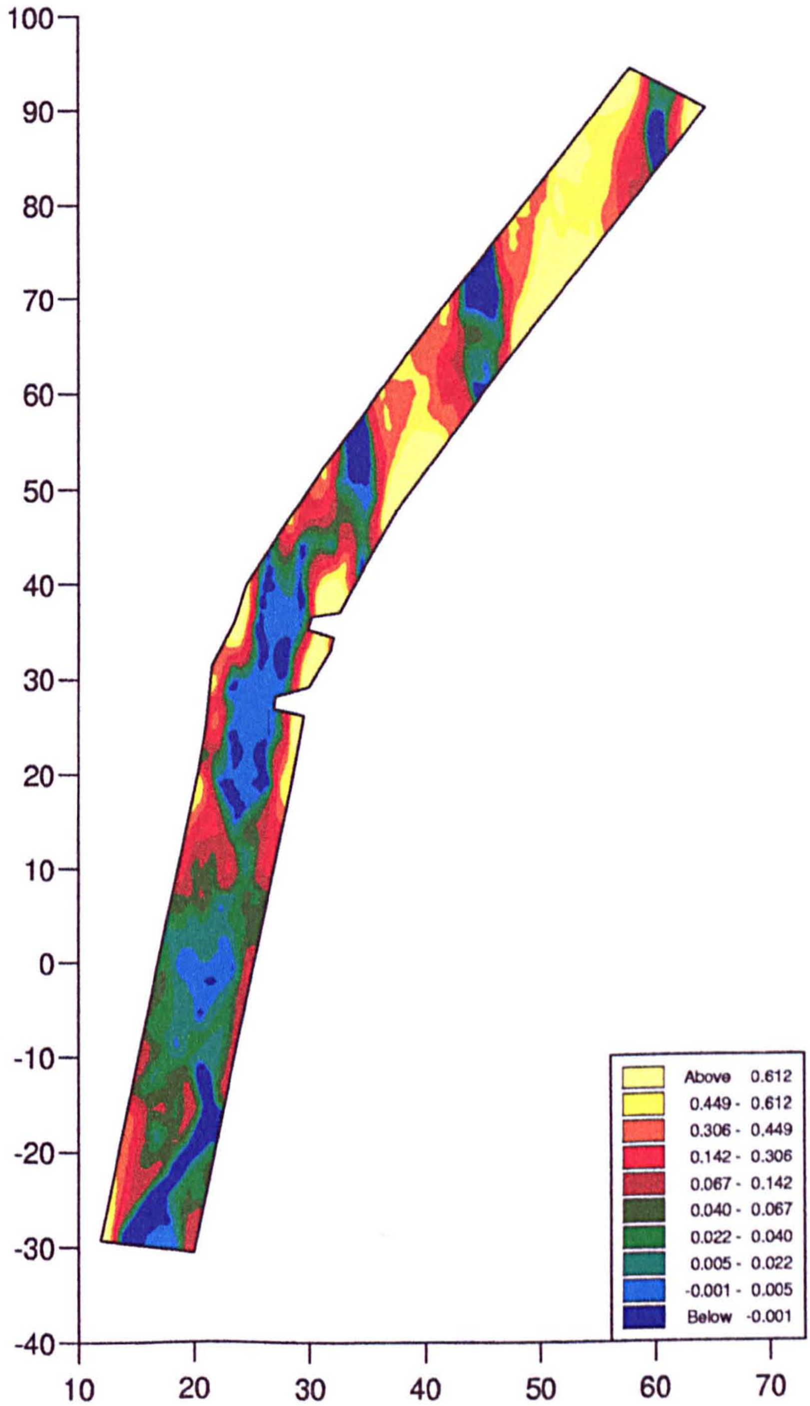
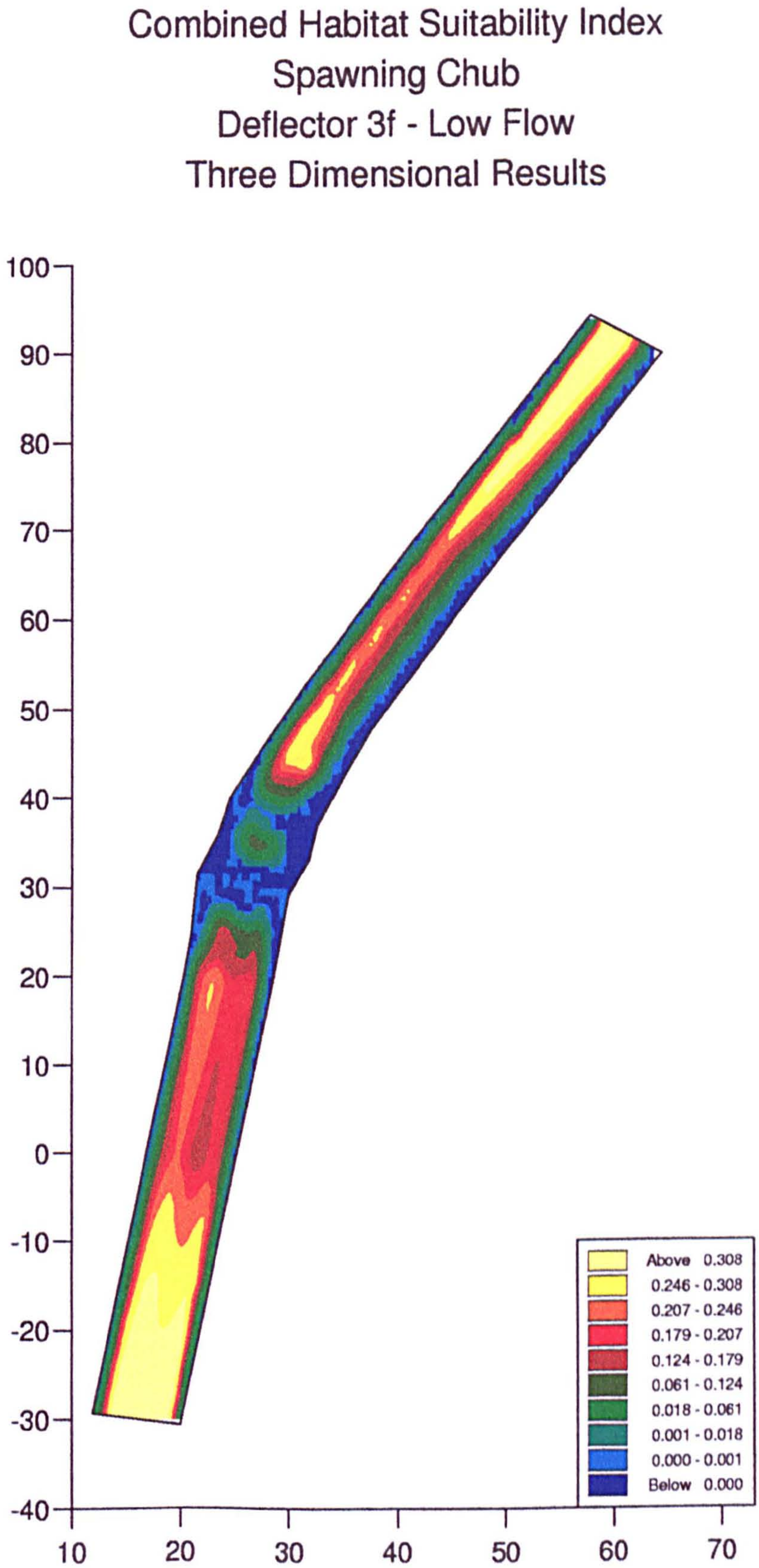


Figure 5.18:



Combined Habitat Suitability Index
Spawning Chub
Deflector 3f - Low Flow
Three Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.308
0.246 - 0.308
0.207 - 0.246
0.179 - 0.207
0.124 - 0.179
0.061 - 0.124
0.018 - 0.061
0.001 - 0.018
0.000 - 0.001
Below 0.000

Figure 5.19:

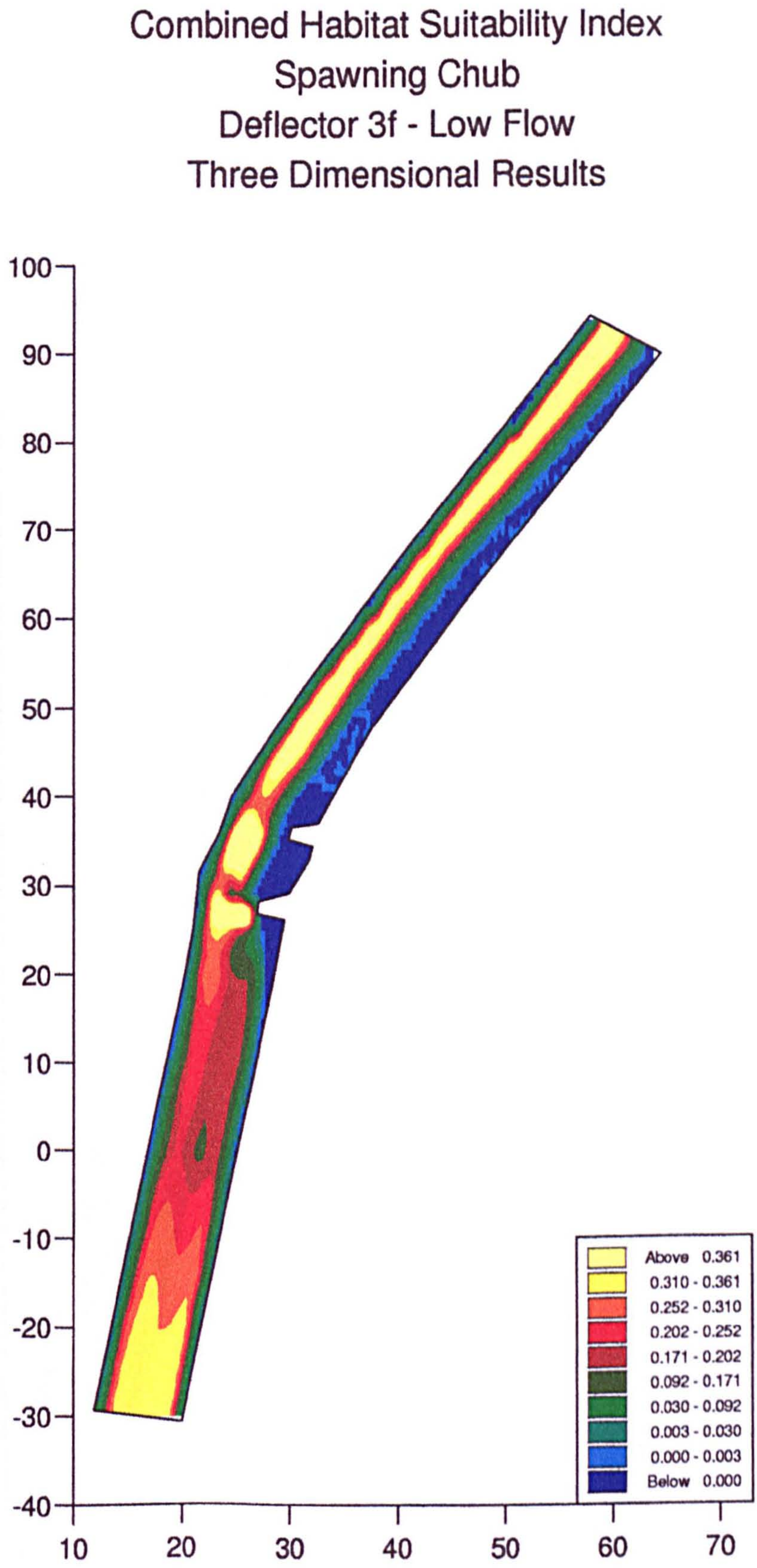
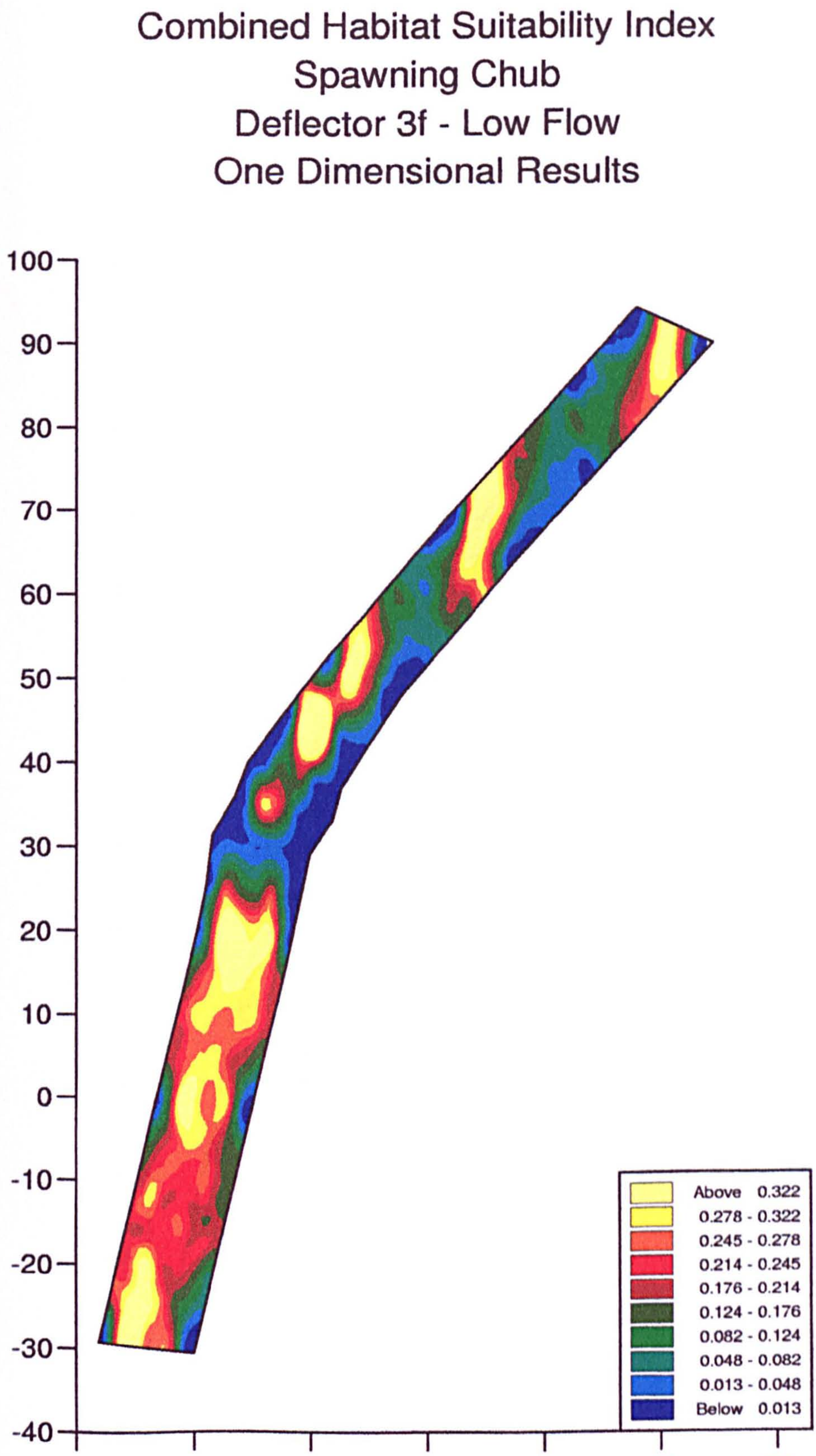

Combined Habitat Suitability Index
Spawning Chub
Deflector 3f - Low Flow
Three Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.361
0.310 - 0.361
0.252 - 0.310
0.202 - 0.252
0.171 - 0.202
0.092 - 0.171
0.030 - 0.092
0.003 - 0.030
0.000 - 0.003
Below 0.000

Figure 5.20:



Combined Habitat Suitability Index
Spawning Chub
Deflector 3f - Low Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.322
0.278 - 0.322
0.245 - 0.278
0.214 - 0.245
0.176 - 0.214
0.124 - 0.176
0.082 - 0.124
0.048 - 0.082
0.013 - 0.048
Below 0.013

Figure 5.21:

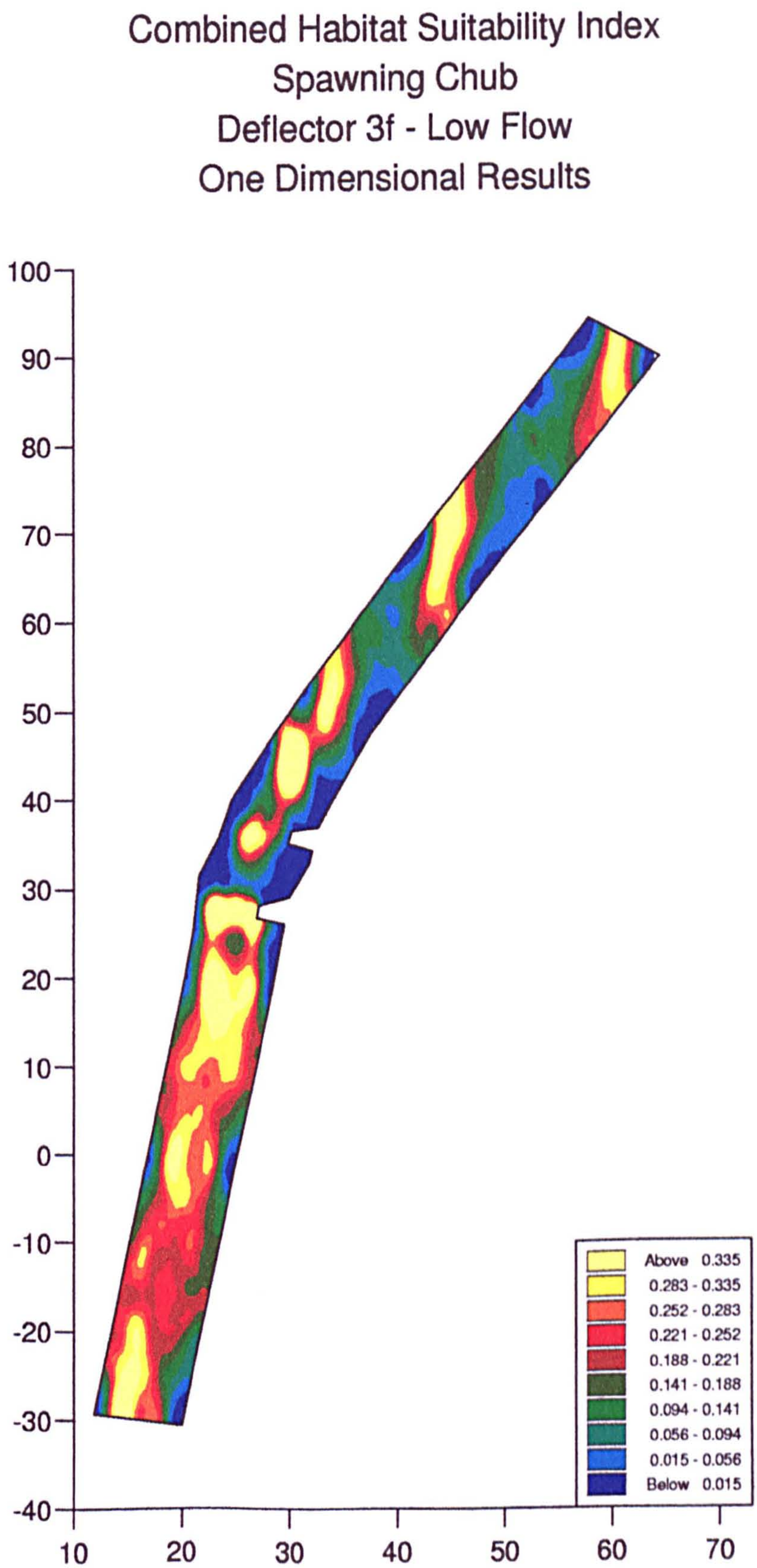

Combined Habitat Suitability Index
Spawning Chub
Deflector 3f - Low Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.335
0.283 - 0.335
0.252 - 0.283
0.221 - 0.252
0.188 - 0.221
0.141 - 0.188
0.094 - 0.141
0.056 - 0.094
0.015 - 0.056
Below 0.015

Figure 5.22:

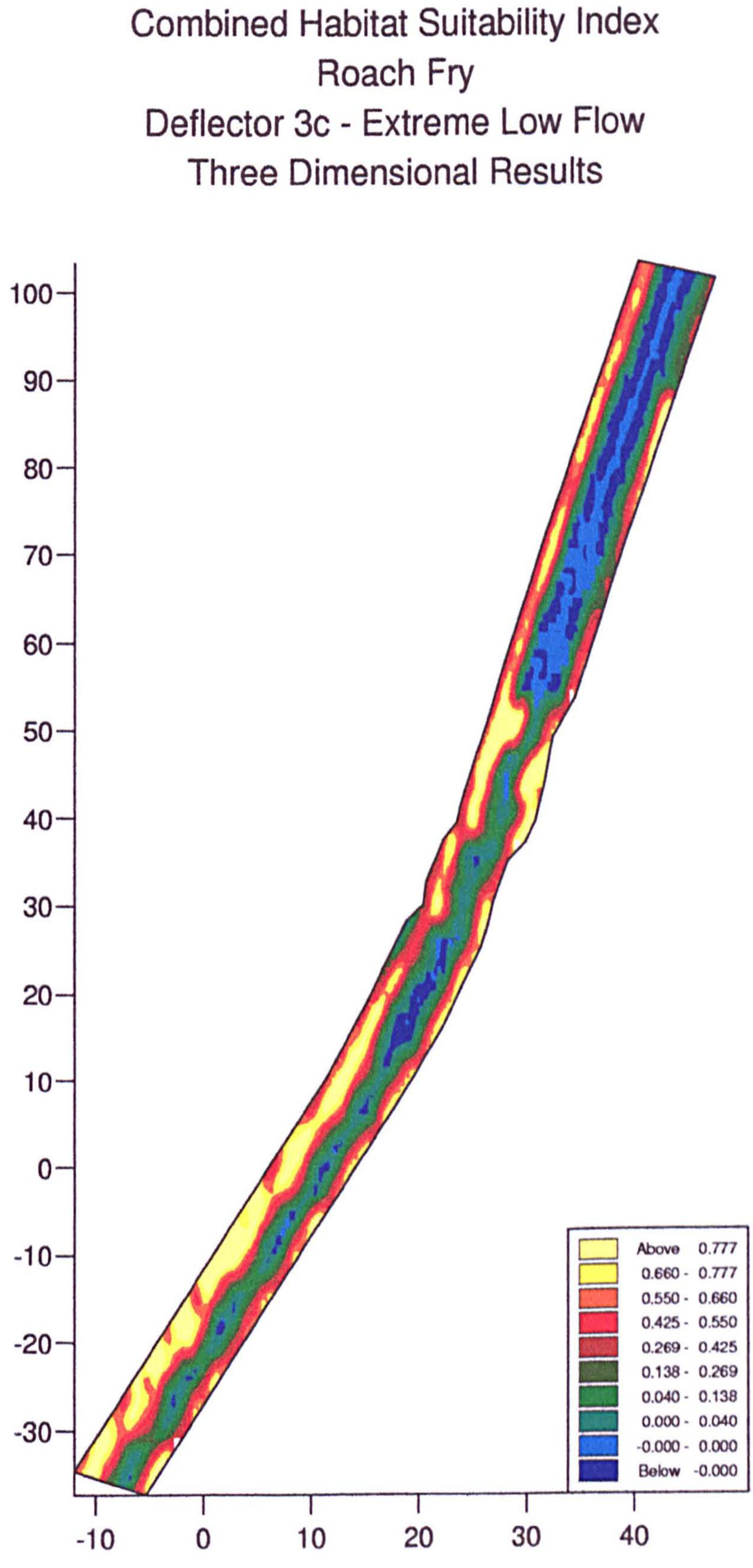

Combined Habitat Suitability Index
Roach Fry
Deflector 3c - Extreme Low Flow
Three Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.777
0.660 - 0.777
0.550 - 0.660
0.425 - 0.550
0.269 - 0.425
0.138 - 0.269
0.040 - 0.138
0.000 - 0.040
-0.000 - 0.000
Below -0.000

Figure 5.23:

Combined Habitat Suitability Index
Roach Fry
Deflector 3c - Extreme Low Flow
Three Dimensional Results

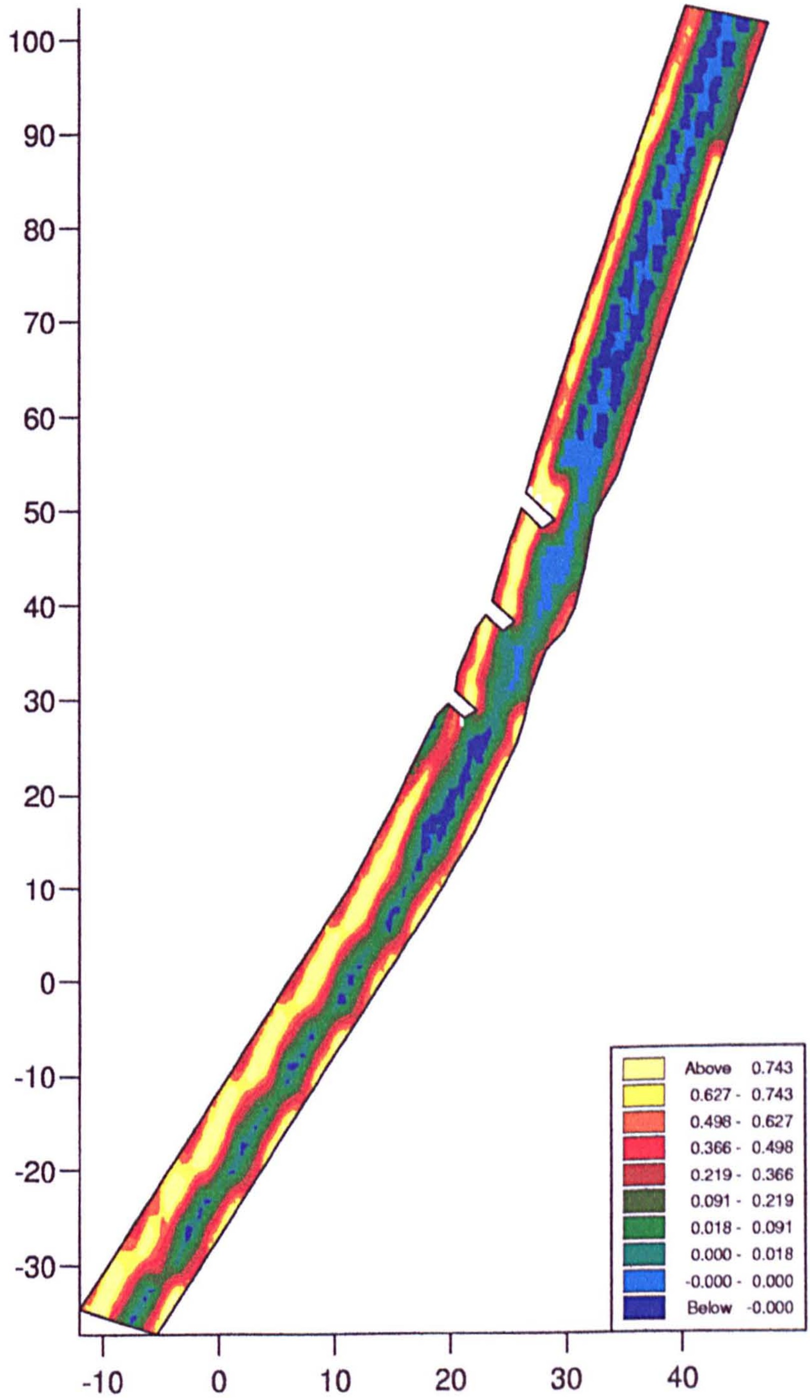
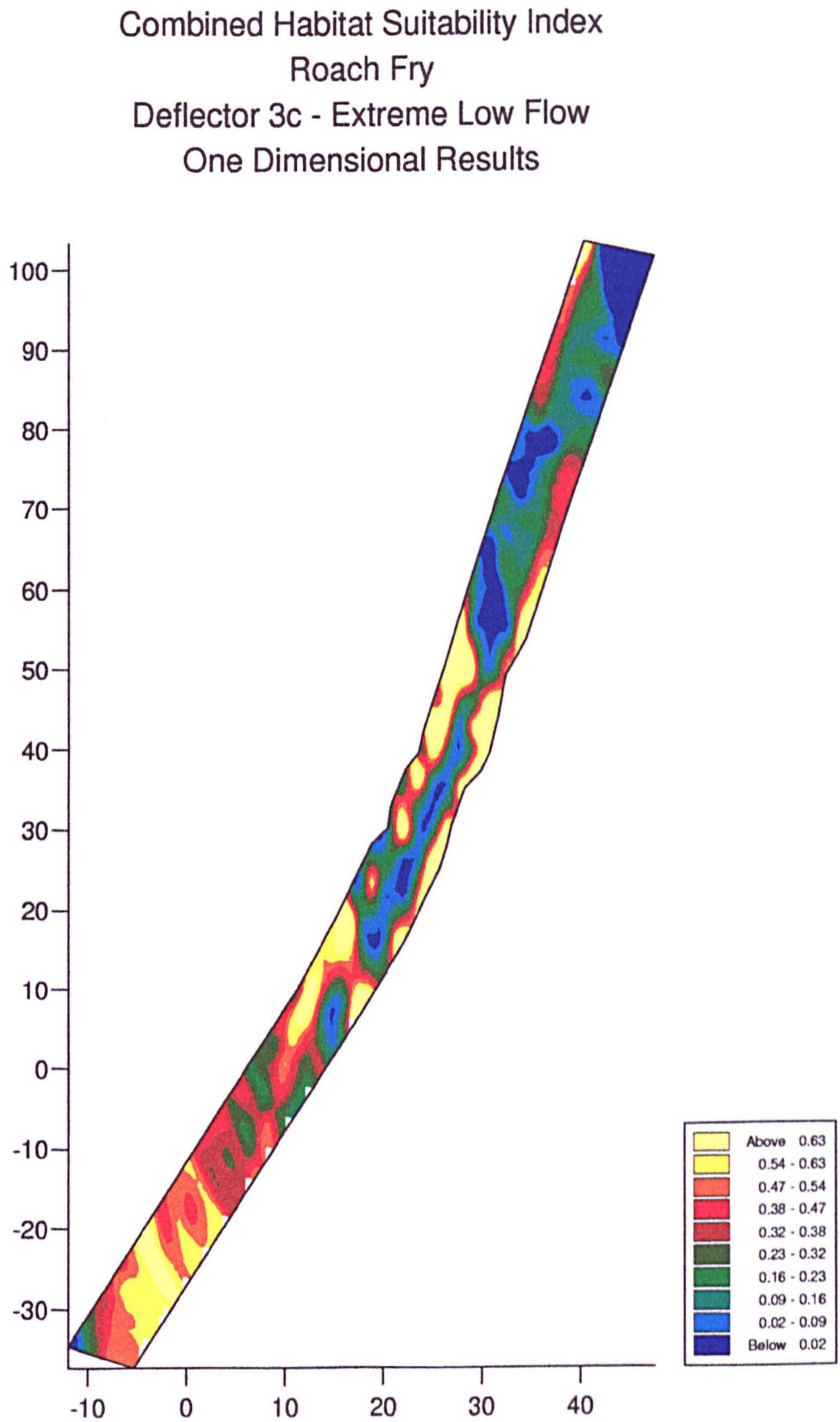


Figure 5.24:



Combined Habitat Suitability Index
Roach Fry
Deflector 3c - Extreme Low Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.63
0.54 - 0.63
0.47 - 0.54
0.38 - 0.47
0.32 - 0.38
0.23 - 0.32
0.16 - 0.23
0.09 - 0.16
0.02 - 0.09
Below 0.02

Figure 5.25:

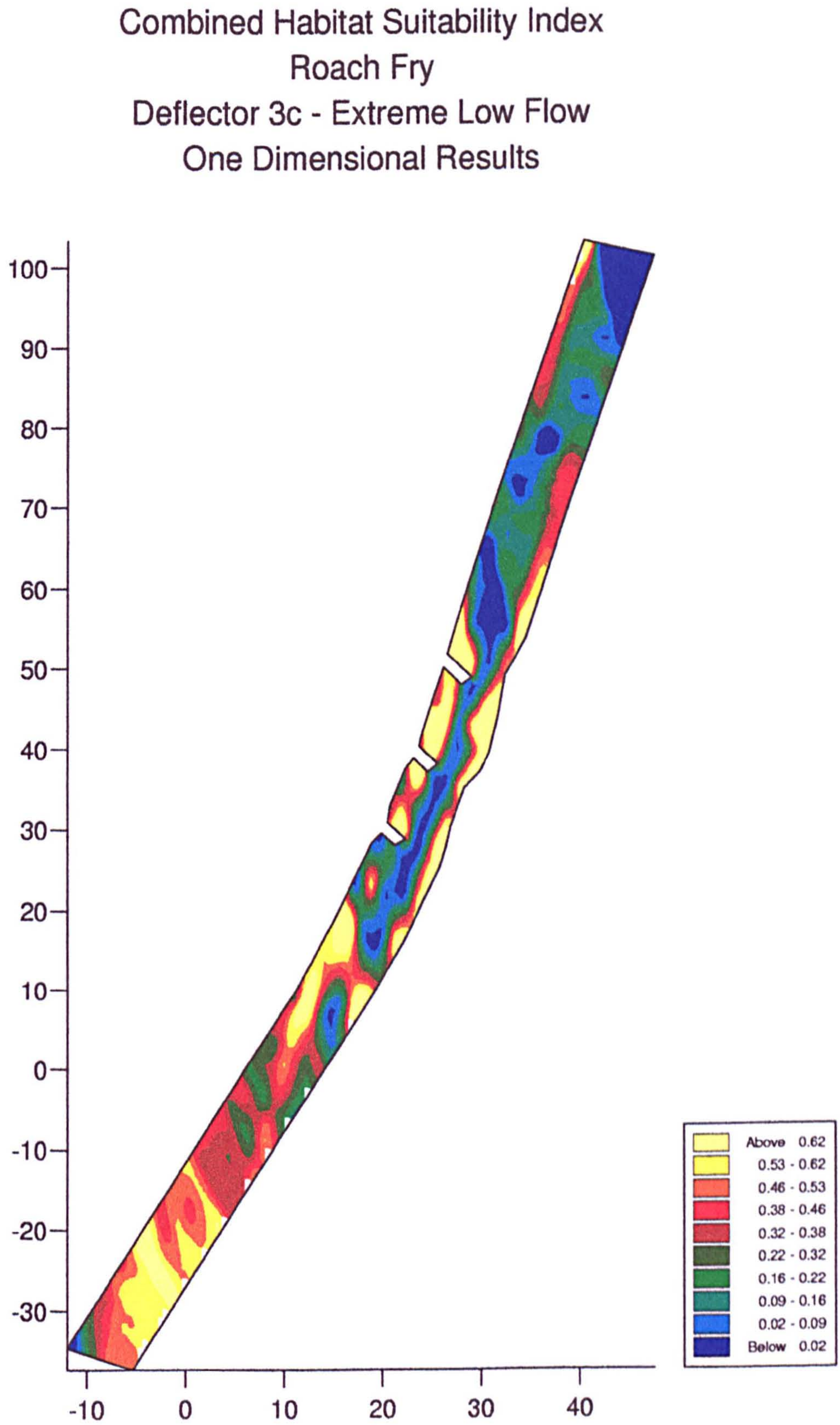

Combined Habitat Suitability Index
Roach Fry
Deflector 3c - Extreme Low Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.62
0.53 - 0.62
0.46 - 0.53
0.38 - 0.46
0.32 - 0.38
0.22 - 0.32
0.16 - 0.22
0.09 - 0.16
0.02 - 0.09
Below 0.02

Figure 5.26:

Combined Habitat Suitability Index
Spawning Chub
Deflector 3c - Extreme Low Flow
Three Dimensional Results

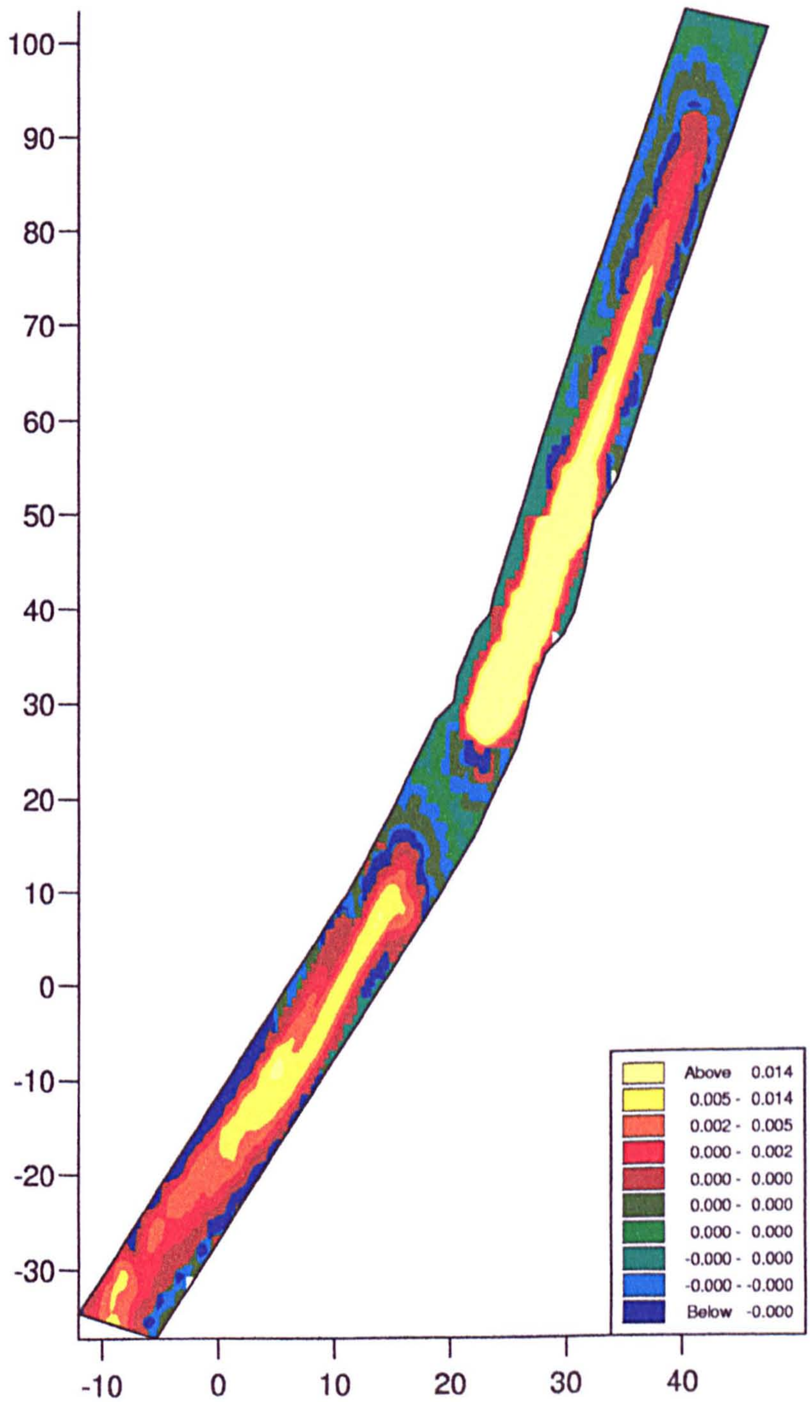


Figure 5.27:

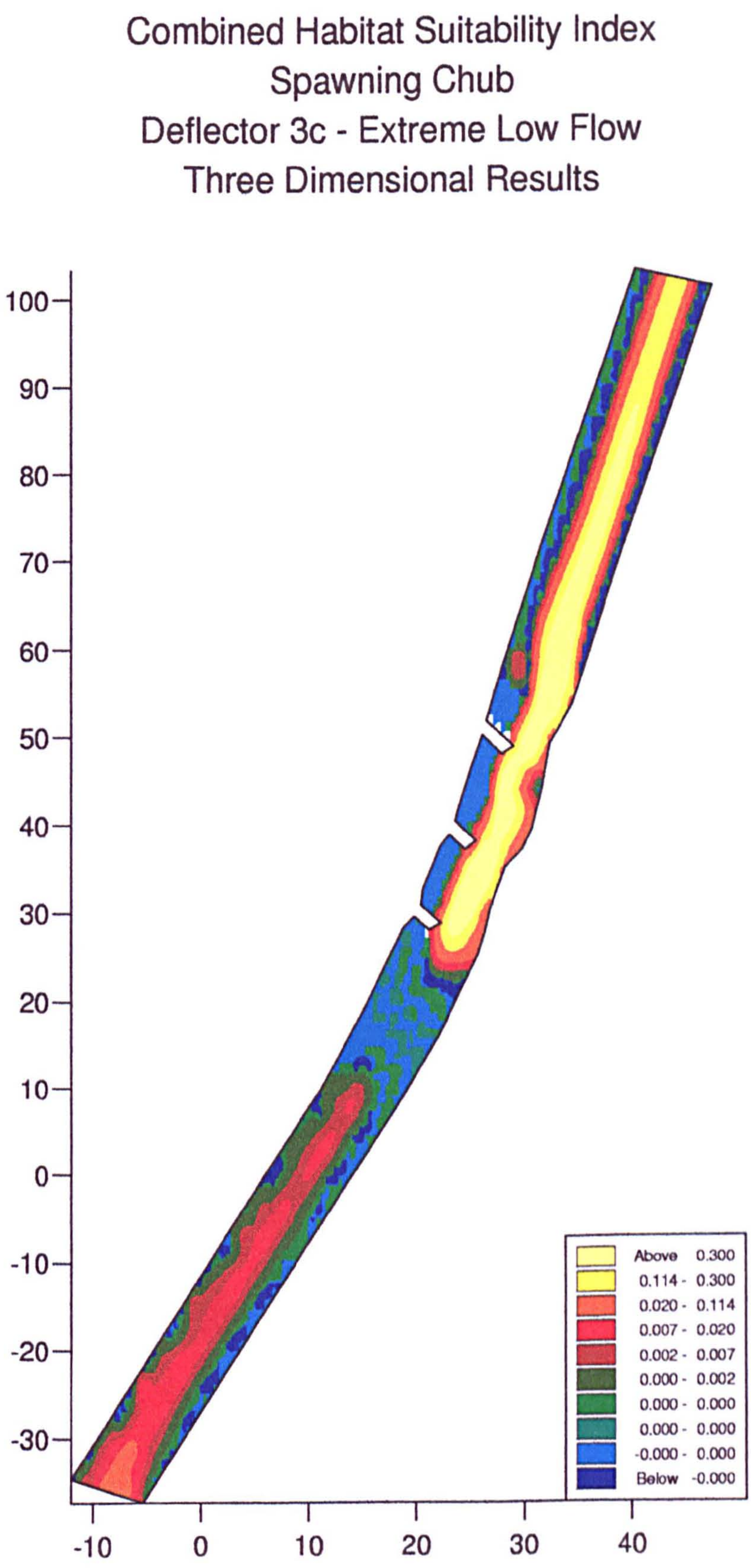

Combined Habitat Suitability Index
Spawning Chub
Deflector 3c - Extreme Low Flow
Three Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.300
0.114 - 0.300
0.020 - 0.114
0.007 - 0.020
0.002 - 0.007
0.000 - 0.002
0.000 - 0.000
0.000 - 0.000
-0.000 - 0.000
Below -0.000

Figure 5.28:

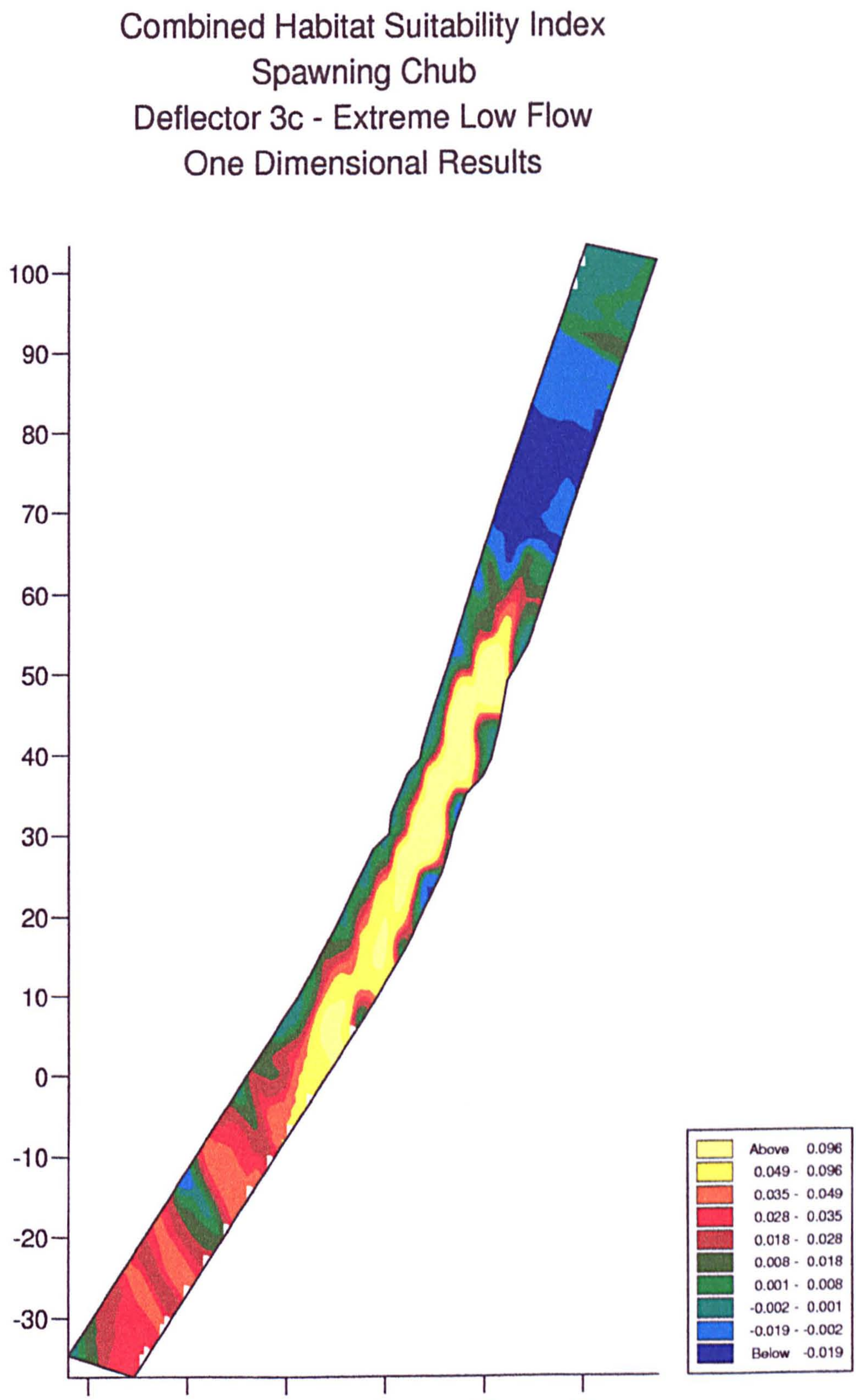
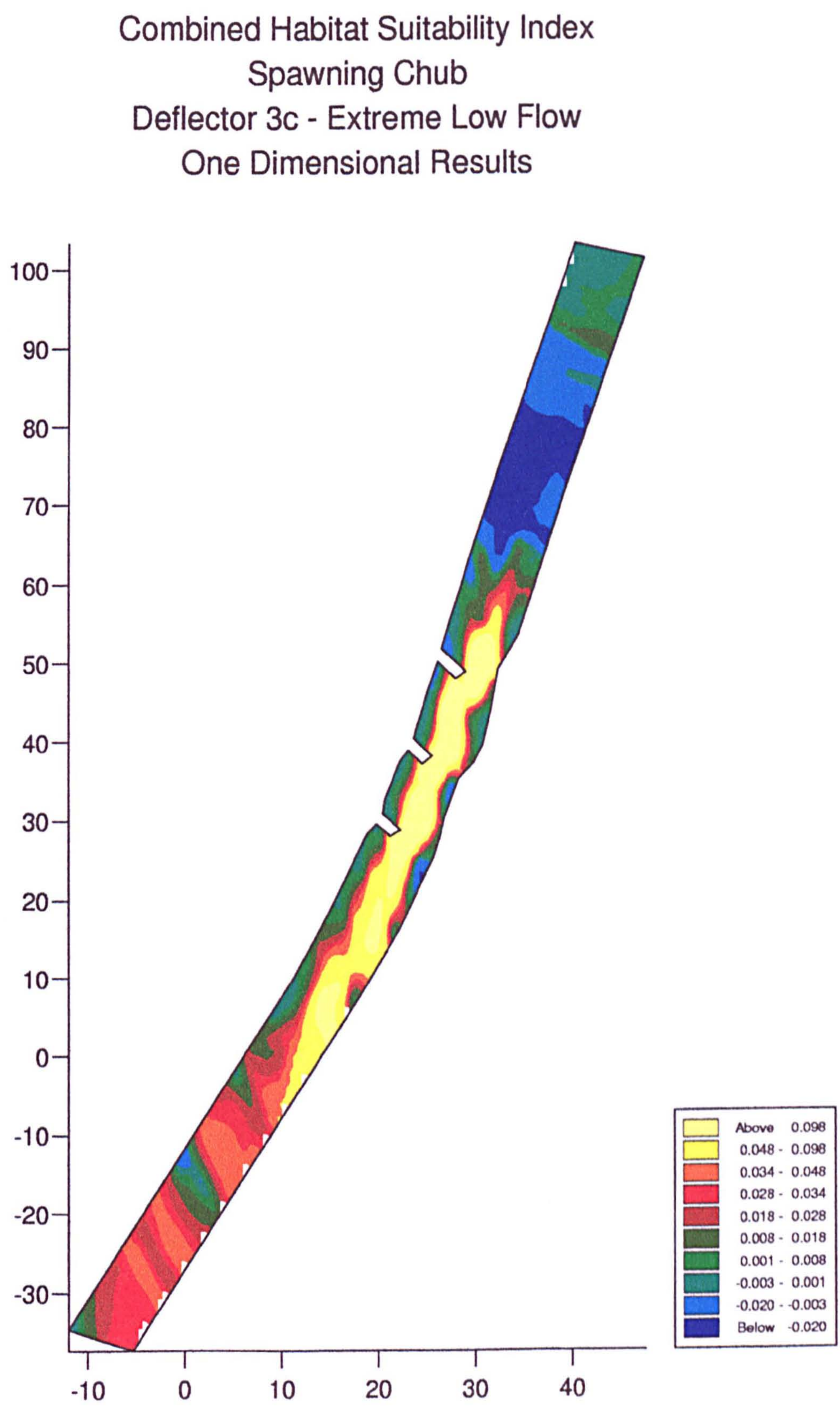

Combined Habitat Suitability Index
Spawning Chub
Deflector 3c - Extreme Low Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.096
0.049 - 0.096
0.035 - 0.049
0.028 - 0.035
0.018 - 0.028
0.008 - 0.018
0.001 - 0.008
-0.002 - 0.001
-0.019 - -0.002
Below -0.019

Figure 5.29:



Combined Habitat Suitability Index
Spawning Chub
Deflector 3c - Extreme Low Flow
One Dimensional Results
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.098
0.048 - 0.098
0.034 - 0.048
0.028 - 0.034
0.018 - 0.028
0.008 - 0.018
0.001 - 0.008
-0.003 - 0.001
-0.020 - -0.003
Below -0.020

Figure 5.30:

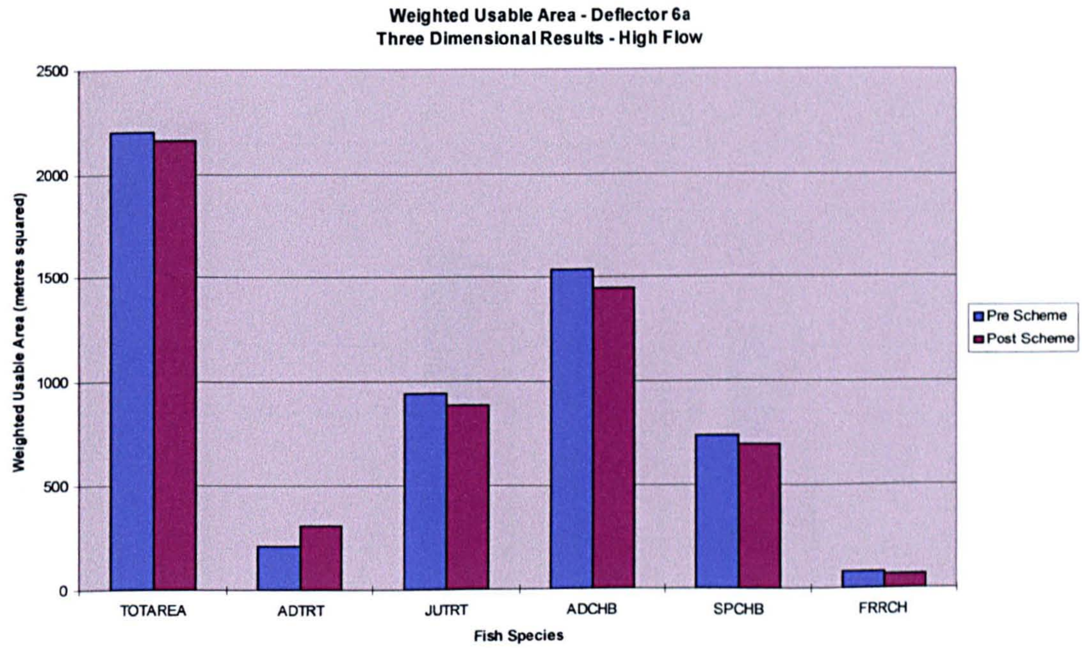


Figure 5.31

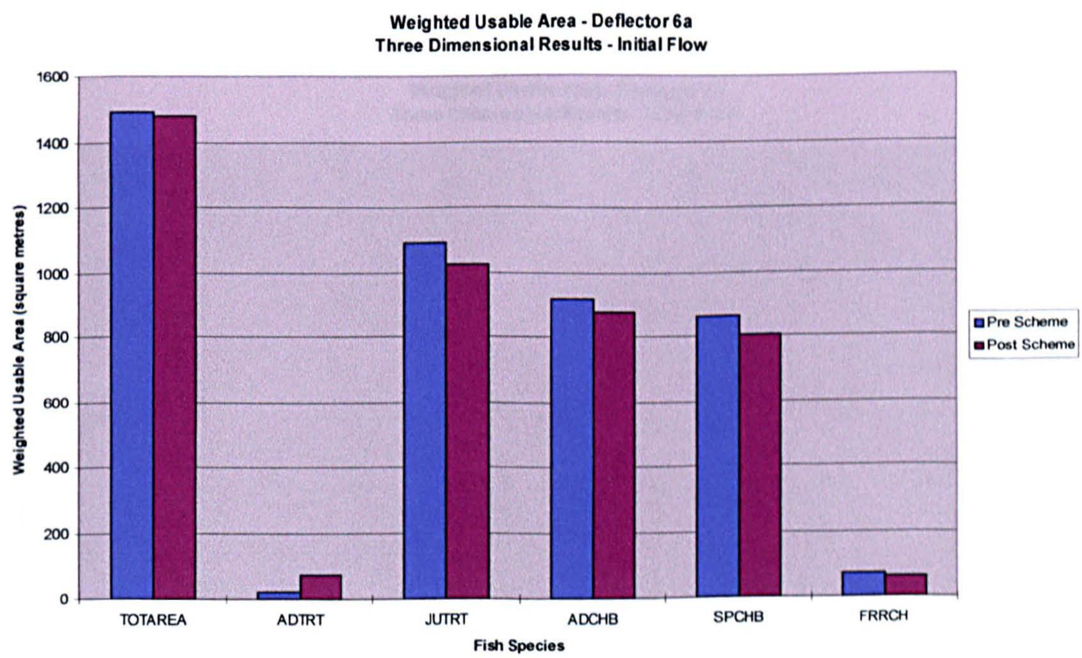


Figure 5.32

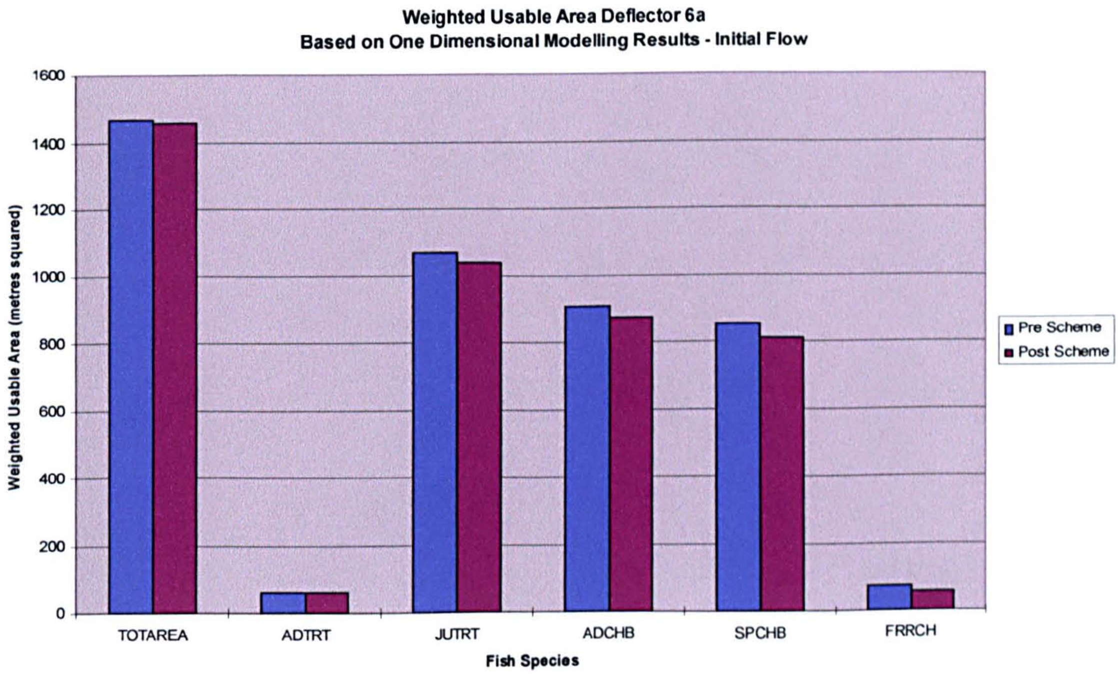


Figure 5.33

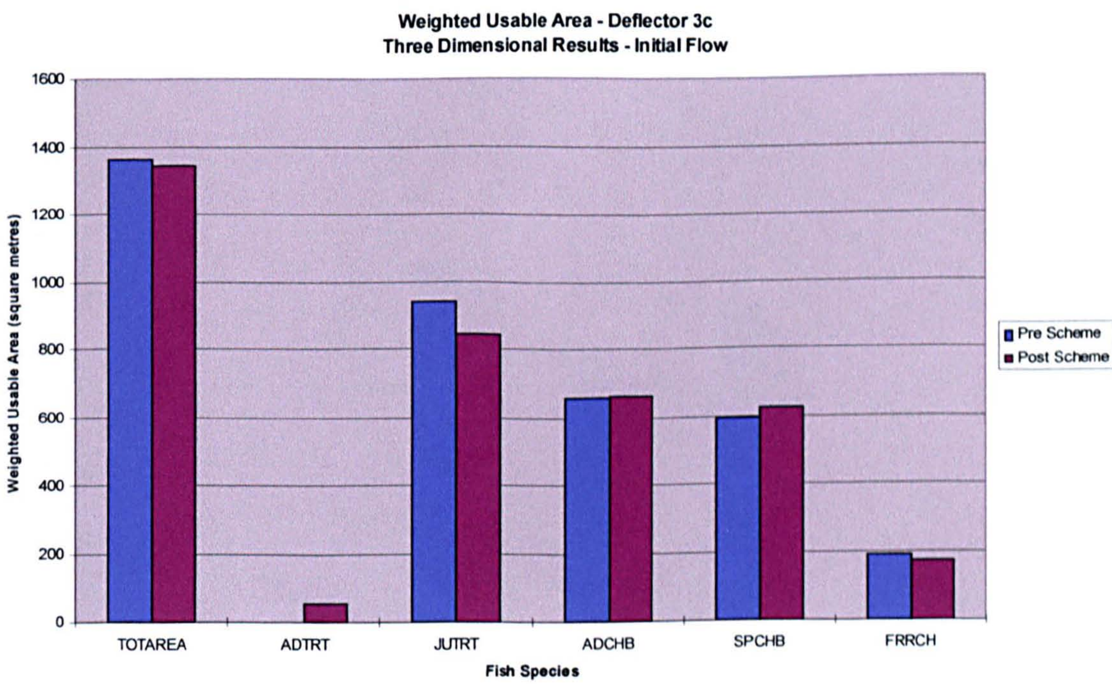


Figure 5.34

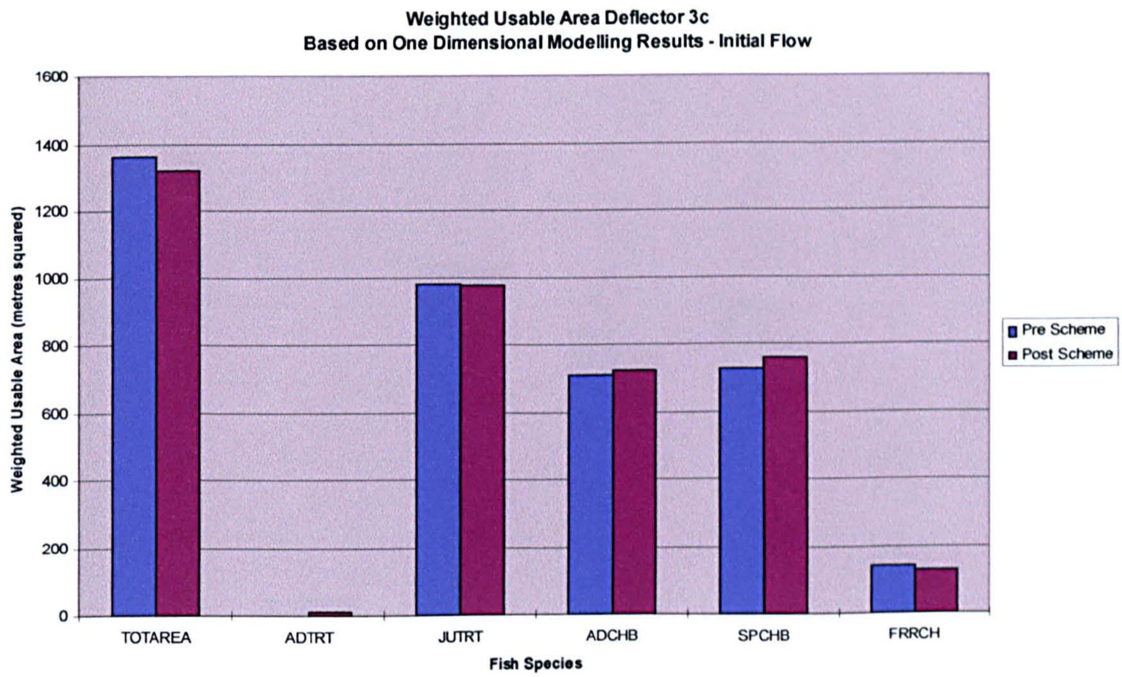


Figure 5.35

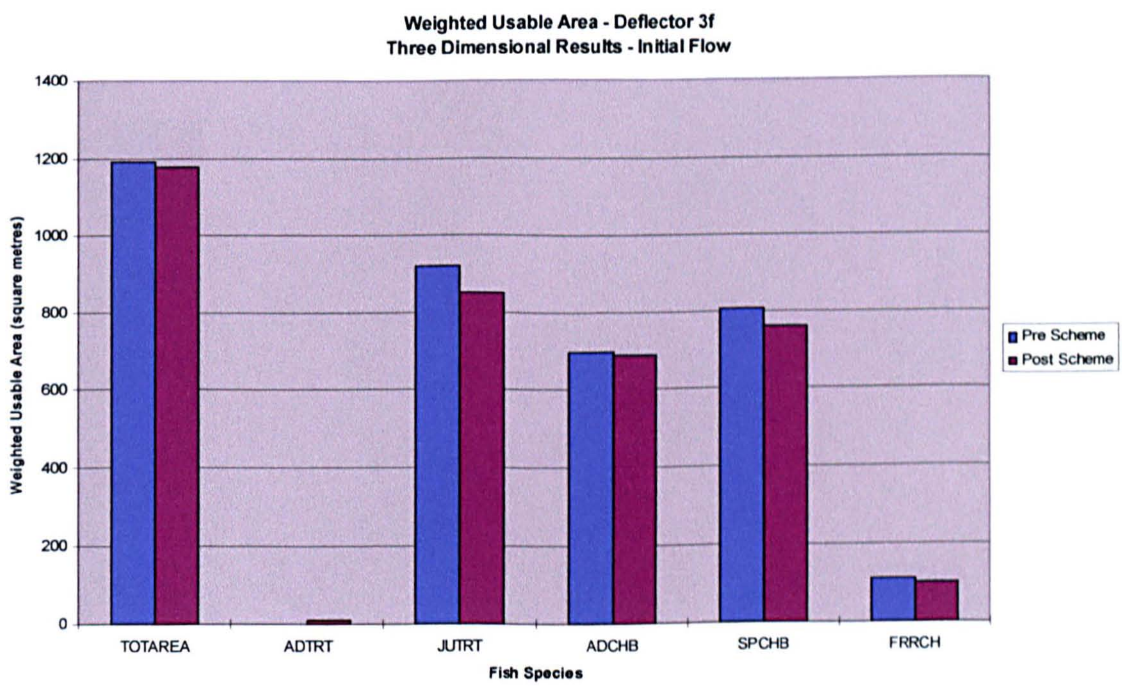


Figure 5.36

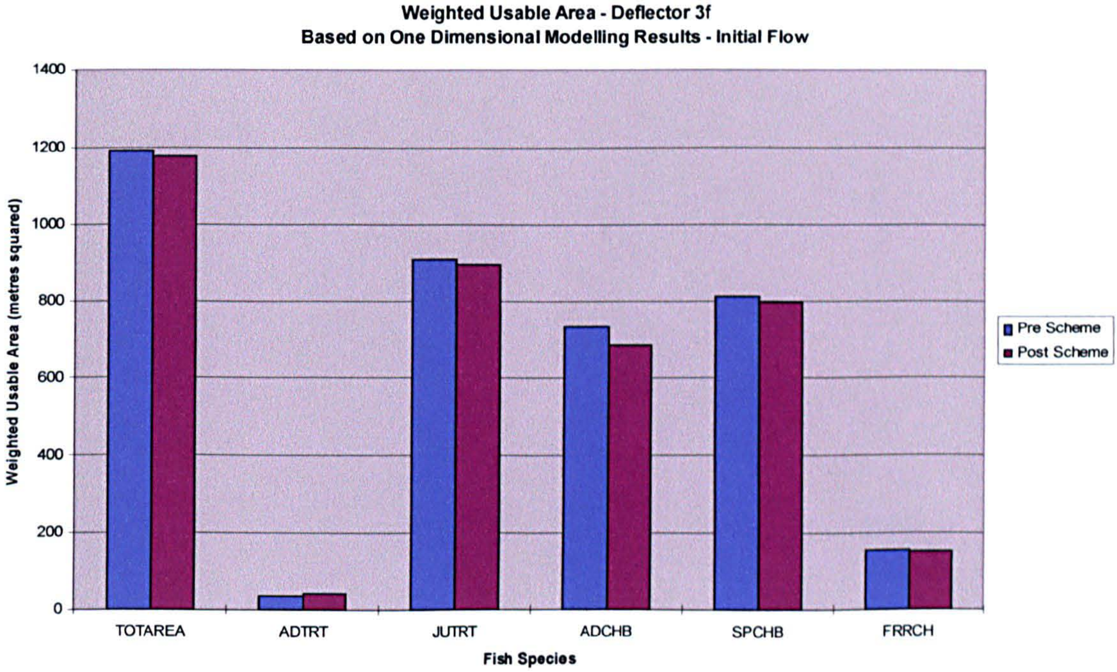


Figure 5.37

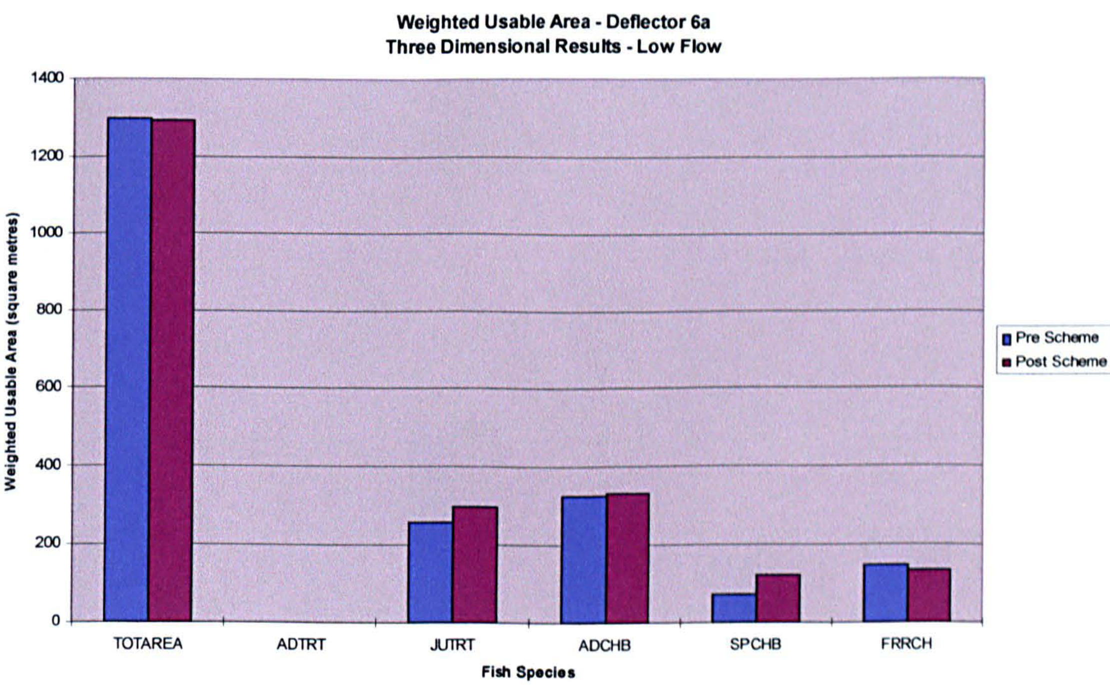


Figure 5.38

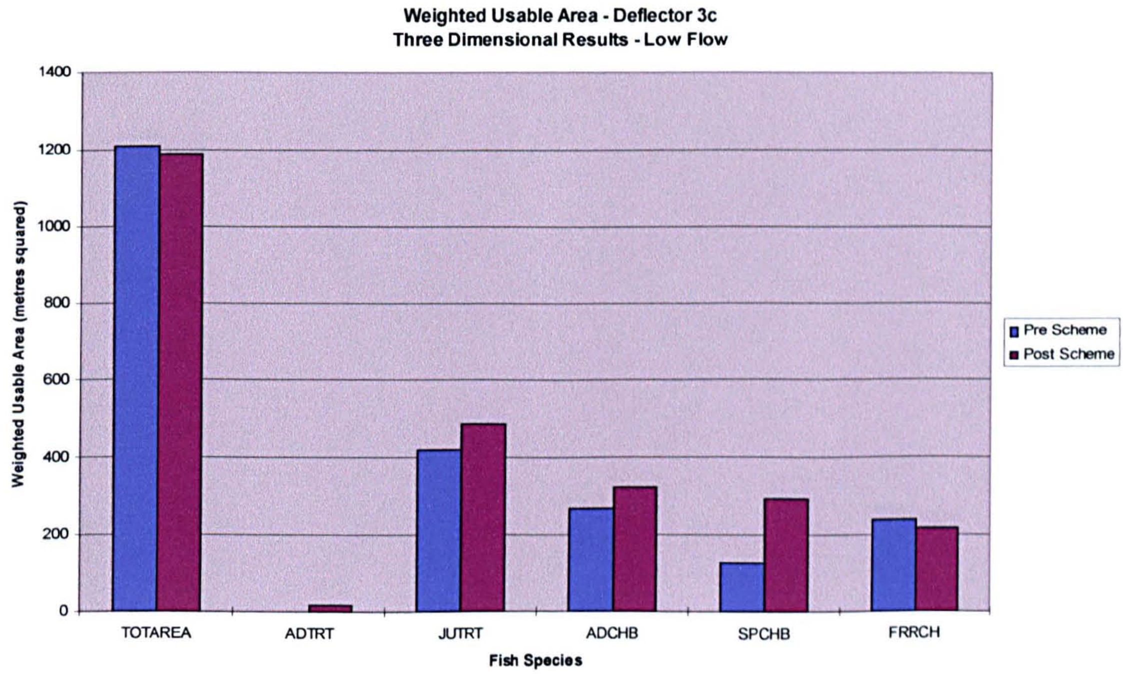


Figure 5.39

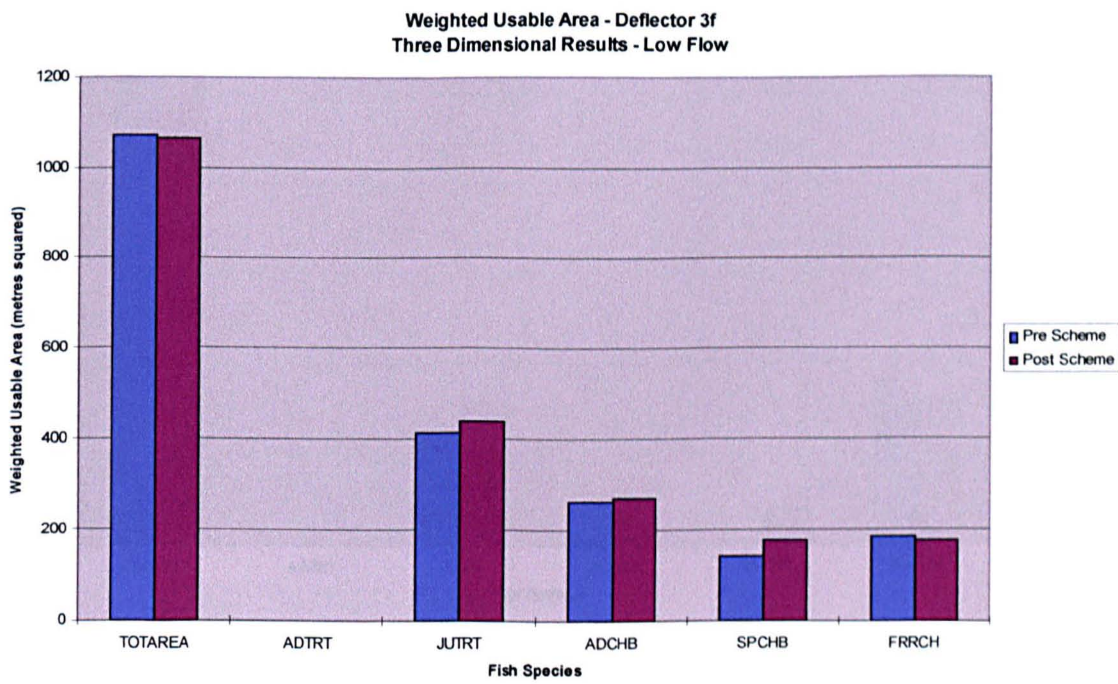


Figure 5.40

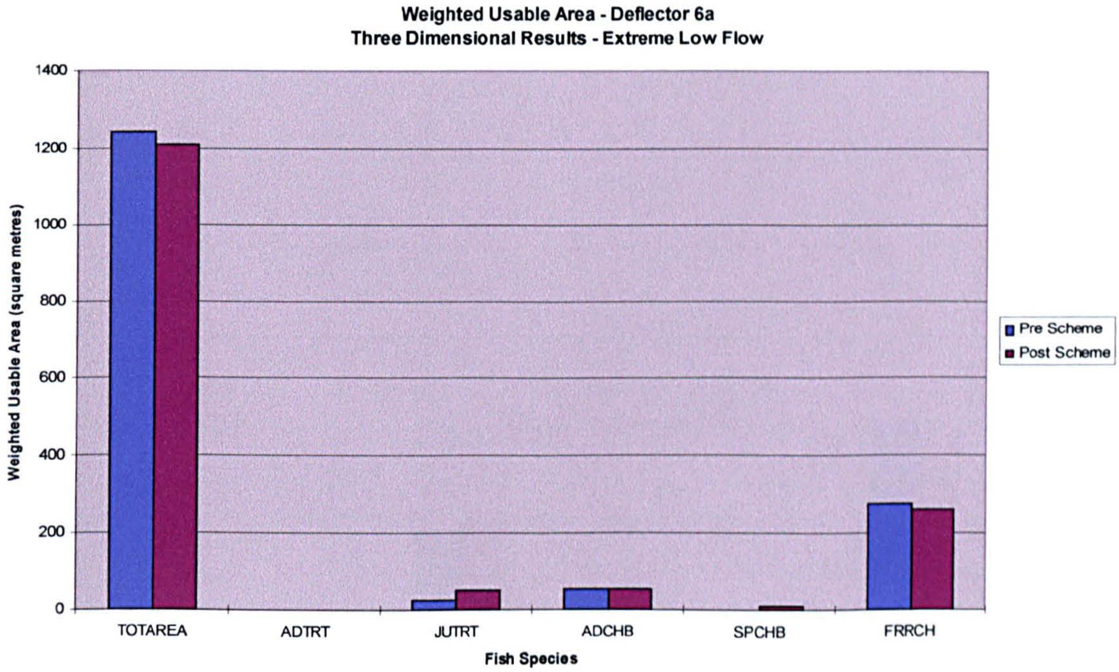


Figure 5.41

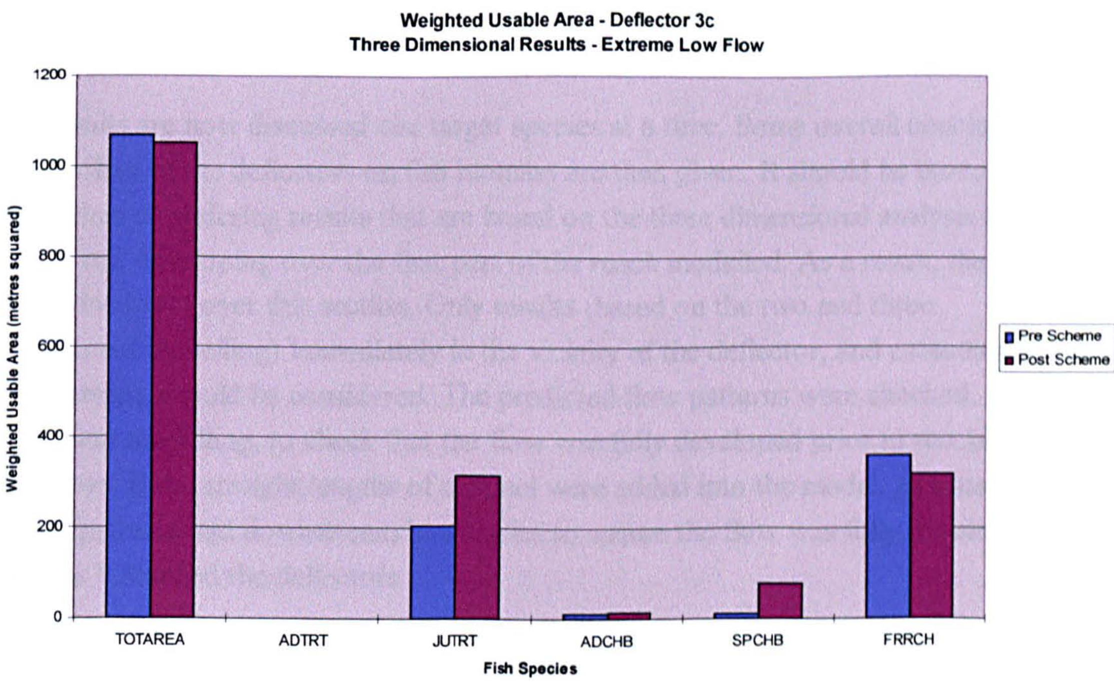


Figure 5.42

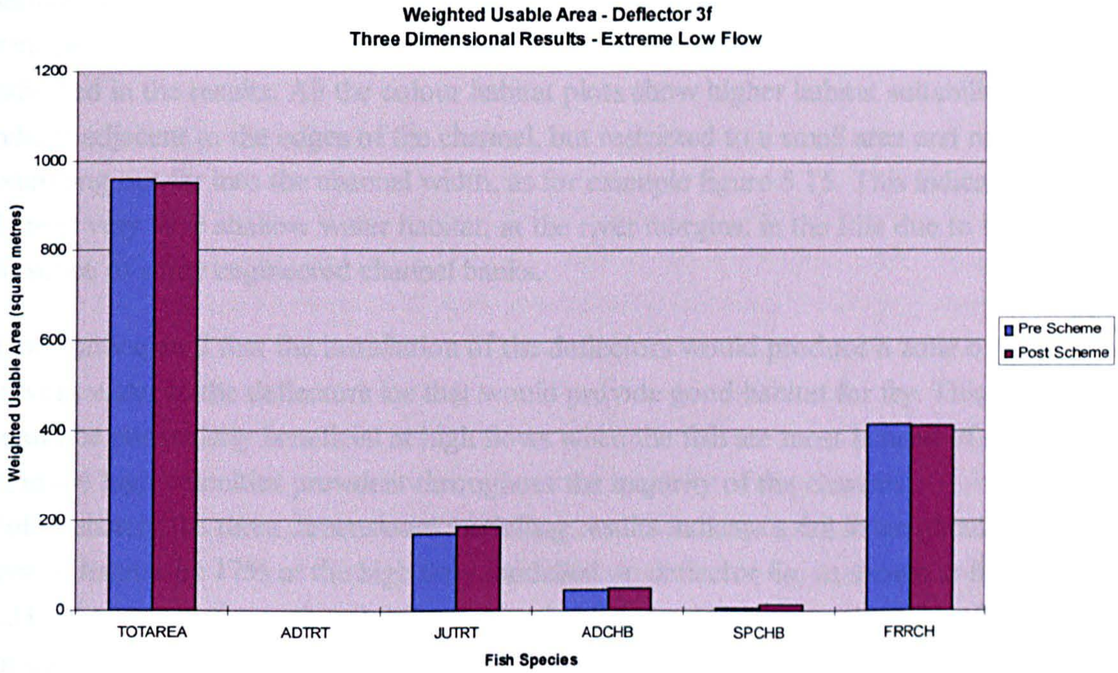


Figure 5.43

5.12. Conclusion

The results are now discussed one target species at a time. Some overall conclusions as to the effect of the deflectors on fish biomass are then given. It should be borne in mind when considering results that are based on the three dimensional analysis that the flow is still developing over the first part of the reach modelled. As a result, the results will be incorrect over this section. Only results (based on the two and three dimensional modelling) immediately in the vicinity of the deflector, and extending downstream, should be considered. The predicted flow patterns were checked, prior to the habitat modelling, to check that the flow was fully developed prior to reaching the deflectors. Extra straight lengths of channel were added into the model, as necessary, at the upstream and downstream boundaries to ensure the flow was fully developed by the time it reached the deflectors.

5.12.1 COMBSIM3D Results

5.12.1.1 Roach Fry

The effect of the deflectors appears to be unexpectedly bad in the case of roach fry. Fry are usually resident in shallower, slow flowing water at the margins of the river. The optimum velocity for roach fry is less than 0.2m/s and the optimum depth is between 0.107 and 0.254m. This is because they are unable to swim against the stronger current in the central part of the channel. This pattern of behaviour is well indicated in the results. All the colour habitat plots show higher habitat suitability indices adjacent to the edges of the channel, but restricted to a small area and not extending out far into the channel width, as for example figure 5.15. This indicates that there is very little shallow water habitat, at the river margins, in the Idle due to the presence of steep engineered channel banks.

It was anticipated that the installation of the deflectors would produce a zone of slow moving water in the deflectors lee that would provide good habitat for fry. This zone would be particularly beneficial at high flows when the fish are most in need of shelter from the high velocities prevalent throughout the majority of the channel. Unfortunately the three dimensional modelling results indicate a fall in weighted usable area of habitat by 17% at the high flow modelled on deflector 6a, as shown in figure 5.31. In addition, the peak suitability index also falls by 44%. This pattern is reflected for each of the three deflectors and for all of the discharges modelled. In each case the weighted usable areas and peak suitability indices either fall or remain approximately constant. This is highlighted in figures 5.15 and 5.16, and 5.23 and 5.24.

At the lower flows the fall in habitat area for fry may not be too significant as there are, in most cases, several hundred metres squared of weighted usable area of habitat present. The fall at the high flow is more significant as not only is the weighted usable area small but the habitat suitability indices are also small (less than 0.1). This suggests that the small amount of habitat that is available consists of relatively inhospitable areas (and not as a small area of relatively good habitat). As a result, the effect of a high discharge on the Idle may be to all but wipe out the fish fry in the river, if suitable areas of habitat are not to be found outside of the sections that have been modelled here. This may well be the single largest reason that the Idle is a poor fishery at present.

The higher velocities that are generated adjacent to the deflector tip are well in excess of those suitable for fry habitat. In addition the highest velocities in the recirculation behind the deflector, as well as the depths at this location, are above those suitable for fry habitat. Once the deflectors had been installed for some months a bank of sediment had accumulated in the lee of each one. Although this situation was not modelled here, it is reasonable to expect that lower velocities would then be evident behind the deflector. This combined with the lower depths (due to the raised height of the bed) would result in an altered habitat behind the deflector that may be much more suitable for fry. Consequently, the installation of the deflectors may have a more beneficial effect on roach fry than would be assumed from the results presented here. These localised areas of shallow water and relatively lower flow velocities may not provide sufficient habitat for the volumes of fry required to support a healthy fishery. More substantial measures may be required to create less steep channel banks, that would generate larger areas of shallow, slack moving water in the river margins.

5.12.1.2 Adult Chub

The immediate effect of the installation of the deflectors on adult chub is considerably more promising than for roach fry. The vast majority of the plotted results show an increase in weighted usable area with a corresponding increase in the maximum value of the combined suitability index. The highest rise in the maximum value of the combined suitability index is 64% and the biggest rise in weighted usable area is 40% (based on the results from COMBSIM3D). The largest increases in habitat availability occur at low and extreme low flow. Habitat availability is most critical for adult chub at the extreme low flow, where there is as little as 10m^2 of weighted usable area available, as can be seen in figures 5.41, 5.42 and 5.43. As a result, the improvements in habitat are seen to be taking place at the discharge where they are most needed.

At each site the deflectors have a positive effect at the extreme low flow by increasing the area of habitat available. At high flow there is a small loss of habitat (6% of WUA) but this is not significant, as there is over 1400 square metres of weighted usable area available.

The extra habitat is generated in the post scheme situation adjacent to the deflector tip and extending downstream. The narrowing of the channel adjacent to the deflector produces a faster flow which is more suitable for adult chub. The area where the flow velocity is increased is not limited to a position adjacent to the deflector, but extends downstream for many tens of metres producing a relatively large improvement in habitat. This can be seen clearly in figures 5.11 and 5.12. Only at the high flow is the velocity which is generated at the deflector tip in excess of that which is the most suitable for adult chub. In brief, the effect of the installation of the deflectors on adult chub should be beneficial, and the effect will be particularly pronounced at the lower range of flows where extra habitat is most required.

Deflector 3c appears to be the most effective in generating extra habitat for adult chub. The peak suitability index is increased by 31%, 64% and 46% at the initial flow, low flow and extreme low flow respectively. The peak suitability index is still very low (0.044) at the extreme low flow though, suggesting that although habitat is improved by the installation of the deflectors it is still in very short supply. Improvements in habitat are less substantial at deflector 3f but the peak suitability index is much higher at 0.223 for the extreme low flow post scheme situation.

5.12.1.3 Spawning Chub

Fairly similar comments to those for adult chub apply to the results for spawning chub, except to say that the findings suggest, in general, even larger habitat improvements. In the most pronounced cases the results from COMBSIM3D suggest the area of habitat is increased by 755% (figure 5.42) and the peak suitability index is increased by 2042% (figure 5.27 and 5.28). Again the largest improvements in habitat are at the low or extremely low flows where improvements are most needed. Even at high flow there is a small improvement in the combined habitat suitability index (but not in WUA).

The extra habitat area is generated in the same fashion as for adult chub, with the higher velocities generated by the narrowing of the channel width producing more suitable conditions. Again this region of more productive habitat extends for many tens of metres downstream of the deflector. This can be seen, for example, in result plots 5.27 and 5.28.

The most suitable flow, of those modelled, for spawning is the initial flow, which was the prevailing discharge when the deflectors were surveyed. Here there is between 600 and 920m^2 of weighted usable area at the three deflector sites. The peak habitat suitability index ranges between 0.995 and 1.0 for the post scheme situation. This suggests that there exists plenty of high quality habitat available for spawning, and this should not be a limiting factor on the numbers of chub present in the river.

At low and extreme low flows there is relatively less available habitat for spawning, but, as fish tend to spawn in the spring to early summer (the coarse fishing season close season extends from mid March to mid June to allow fish to spawn) the flows would not be expected to fall to these low levels for any extended period of time.

Again it is deflector 3c that produces the largest improvements in habitat. The peak suitability index is increased by 36%, 288% and 2042% at the initial flow (figure 5.3 and 5.4), low flow and extreme low flow (figure 5.27 and 5.28) respectively. At the initial flow level the habitat suitability indices are already high for spawning chub.

No fish surveys have been carried out to determine the effect of the deflectors on biomass. It may take several years for the deflectors effect on aquatic species to fully take effect. The results produced here are encouraging in terms of adult and spawning chub, but the decreases in available habitat for roach fry are a big cause for concern.

5.12.2 COMBSIM1D Results

The results from COMBSIM3D give a more accurate representation than COMBSIM1D of the habitat improvements that the deflectors give rise to. This is because the 3D analysis solves the flow field more accurately than is possible using the panel velocities from ISIS. As a consequence, the results from 3D have been used in the discussion above, in order to quantify the likely effect of the deflectors on fish habitat. It now remains to discuss how successful the 1D results are in reproducing the effects described by the 3D software.

In the large majority of cases, the predictions of weighted usable area prior to the installation of the deflectors are very similar in the 1D and 3D cases. This can be seen, for example, by comparing figures 5.36 and 5.37. The peak values of the suitability indices are somewhat less similar as can be seen by comparing figures 5.3 and 5.5, for example. This tends to indicate that although the 1D habitat predictions compare well with the 3D ones overall, in a few areas there may be some pronounced areas of difference. These localised areas, where the two models predict slightly different velocities, produce a correspondingly higher or lower suitability index. The differences between the three dimensional and one dimensional results can only be attributed to differences in the computed flow field, as the depth and substrate information input to COMBSIM3D and COMBSIM1D are identical.

The major differences in results between the three dimensional and one dimensional habitat predictions occur after the deflectors have been installed. Here it can be seen that the three dimensional results identify much more considerable changes in both available habitat area and in peak suitability index. Changes in available habitat area using COMBSIM3D are as much as 755%, and are typically between 7% and 11%. Changes in peak suitability index are as high as 2042% and are typically between 5% and 30%.

Using COMBSIM1D the peak increase in WUA is 31% and the largest increase in the peak suitability index is 600%. Typical changes in both WUA and peak suitability index are in the range of 1-6%. The difference in WUA results can be explained by the fact that the three dimensional model solves the full three dimensional momentum equations whilst the one dimensional model does not. To highlight this point, consider the way in which the two models calculate the velocities in the region of the deflector. The one dimensional model will increase the velocities at the deflector in response to the narrowing of the channel cross section. The flow velocities may also be higher downstream of the deflector (i.e. at locations in which the channel width has returned to a similar dimension as upstream of the deflector) as the model solves the momentum equation in one dimension. However, once the program comes to calculate the panel velocities behind the deflector the overall increase in velocity for the entire cross section will become averaged out over all of the panels. This is due to the fact that the model simply calculates the panel velocities by taking the overall cross section conveyance, and the conveyance in each panel, and determines the average velocity in each panel in proportion to the conveyance in that panel. In other words, each panel is treated in the same way irrespective of its position across the cross section, and irrespective of whether it is immediately in the lee of the deflector (geometrically behind it) or downstream of the gap between the deflector and the opposite bank (where higher velocities would be expected).

In contrast, by solving the three momentum equations, the plume of high velocity which issues from the gap between the deflector and the bank is accurately traced downstream by the three dimensional model. This effect can clearly be seen in many of the three dimensional results plots, but perhaps most clearly in figures 5.3 and 5.4, where a band of higher habitat suitability index (which correspond to the zone of faster moving water) can be seen extending for a considerable distance downstream of the deflector. This effect is completely absent in the corresponding one dimensional results plots 5.5 and 5.6, where the higher velocity downstream of the deflector becomes averaged out over the entire cross section. This localised plume of high velocity and relatively high suitability index accounts for the greater increase in WUA observed in the three dimensional results.

The effect of the recirculation that occurs in between and downstream of the deflectors is also entirely lost in the one dimensional results. In plot 5.4, which is based on COMBSIM3D, low suitability values are predicted between and downstream of the deflector due to the presence of relatively lower velocity recirculations in these areas, relative to the higher flow speeds in the main part of the channel. This can also be seen in plot 5.8, for example. In plots 5.6 and 5.10, from COMBSIM1D, higher suitabilities are predicted in these areas as the recirculations are entirely absent. It would be possible to improve the 1D predictions by calibrating the predicted panel velocities in both the pre and post scheme situation. However, of course it would not be possible to calibrate post scheme velocities at the design stage. This would only be possible after the deflectors have been installed.

The difference in the increase in peak suitability index between the 1D and 3D results can also be explained by the way that velocities are calculated in the two packages. The highest suitability index for two out of the three target species studied (namely adult chub and spawning chub) occurs in a region extending from immediately adjacent to the deflector tip and extending a short distance downstream. This highest suitability index corresponds to the location at which the highest velocity in the reach occurs (since the velocities prior to the installation of the deflectors at the vast majority of flows are below that for optimum habitat). Adjacent to the deflector tip there can be as few as six panels across the channel width in the one dimensional model. In addition, the distance downstream to the next panel can be several metres. In the three dimensional model, each panel across the channel width is further subdivided (typically into between 3 and 5 cells). The distance downstream to the next cross section is also subdivided (typically into between 5 and 7 cells).

The panel velocities that the one dimensional model predicts are average values for that portion of the cross section. If the depth is less at one side of a panel than at the other this would in reality produce a faster flow at each side of the panel. In the one dimensional model this effect is smeared out as the velocities are averages for each cell. Thus velocity peaks are likely to be removed by the averaging algorithm. At the deflector, as few as six panel velocities can be calculated. In the three dimensional model a minimum of eighteen velocities (six panels times three cells per panel) are calculated (which have not been averaged out) so the calculated flow field is more detailed and will represent peaks and troughs in velocity across the section in more detail. The one dimensional results could perhaps be improved by introducing more panels across the width of each section. This could be done by taking more survey levels across each section or by artificially interpolating extra panels from the existing data. At present the position and size of the panels simply correspond to locations across the sections at which levels were surveyed.

As has been already observed, the peak velocity may also occur not at a section immediately adjacent to the deflector tip but at a short distance downstream. The one dimensional model will be unable to detect this, partly because it does not solve the full set of momentum equations and partly because its cross sections are spaced apart by several metres. The point at which the maximum velocity occurs is not likely to coincide with the location of a cross section in the one dimensional model. The three dimensional model, with its further discretisation of cells between cross sections and the fact that it solves the three dimensional momentum equations can track the momentum in detail to the point at which the maximum velocity is produced, wherever that may occur (i.e. it is not just limited to calculating velocities at cross sections as the one dimension model is).

A further consequence of the fact that the two and three dimensional models are able to calculate the flow field more accurately is that the results plots of habitat suitability index look considerably smoother. Habitat plots from the one dimensional program look disordered or chaotic and appear to be less intuitively logical. As the one dimensional program is unable to accurately locate the highest velocities in each cross section, the position of the highest velocity corresponds to the panel with the greatest depth (and thus the greatest conveyance). Thus, as the flow progresses downstream the highest velocity (and highest suitability index in most cases) can switch rapidly from one side of the channel to the other in a way that is not realistically possible when considering the momentum equations in two or three dimensions. As a result, in the habitat plots for the one dimensional results the highest suitability index moves about in the channel, as the flow goes downstream, to the position of the greatest depth. In the two and three dimensional results the path of the highest suitability index follows the path of the highest velocity, as predicted by the momentum equations.

The combined result of this is that although the one dimensional model makes reasonable habitat predictions prior to the installation of the deflectors, it under predicts both the increase in habitat area and the location and suitability of the best habitat after the deflectors have been installed. One dimensional predictions may be improved by introducing more data into the model but results can never match those from three dimensional models for accuracy. Calibration of one dimensional velocity predictions would definitely improve predictions but cannot be undertaken until the scheme has been constructed. The benefits of a two or three dimensional approach therefore appear to be quite considerable.

5.13. Discussion

To model the entire effect of the installation of the deflectors on aquatic species would have required the consideration of a much larger number of discharges. In addition, every life stage for each of the species of fish found in the river (and those that potentially may colonise it if conditions were suitable) would have had to be included. The resulting number of shaded colour plots and usable areas of habitat graphs would have been very considerable. To analyse and draw overall conclusions from such a vast bank of results would have been very problematical. Instead, the results presented here are manageable but have their limitations in that they relate to only four discharges and three life stages of two target species.

As has been commented on above, the true measure of the effect of the deflectors can only be ascertained by modelling them with the altered channel geometry that they produce (i.e. the scour pool at the deflector tip and the bank of sediment behind the deflector). This work is beyond the scope of this project, but it is referred to in the section on future research needs in the conclusion to this thesis.

The approach to habitat modelling in this thesis uses the concept of IFIM introduced earlier in the chapter. As a result, the results that have been obtained are open to the criticisms that are normally levelled at this type of approach, and these have also been outlined earlier in the chapter. Chief among these criticisms would be: that the interrelation of the governing variables is not taken into account, the limitations inherent in the use of preference curves and the fact that the method does not include the effects of biological interactions.

Despite the numerous criticisms aimed at it, IFIM is a very useful tool in providing a standard approach to problems relating to habitat change. Many papers have been published highlighting the inadequacies of the approach and the variability in results that can be obtained. However, there are fewer papers that outline an approach that is clearly based on better ecological principles, or that will yield more reliable and consistent results. Perhaps the relatively new methods behind energistic models (outlined in the earlier literature review) may prove to be an exception to this, but they are yet to be widely tested. Bird states that 'there are presently no superior alternatives to IFIM'.

The fundamental problem with IFIM is that the results that it yields are not predictions of fish biomass, but instead an estimate of the available habitat. (Gore and Nestler, 1988) quote the comments provided by Bovee who suggests that WUA-discharge relations are indicators of the carrying capacity of the stream reach relative to stream flow. In effect this is a disclaimer to say that even if an accurate and detailed analysis using IFIM is undertaken, the results cannot be relied upon to give an accurate indication of habitat improvement. WUA predictions cannot be correlated with fish biomass because of the current limitations in the approach mentioned above and discussed in detail earlier. One metre squared of usable area may contain one of the target species or fifty. It has been suggested that to include additional factors would be to reduce the general applicability of the method. (Gore and Nestler, 1988) quote the maxim provided by Levins that 'population models cannot simultaneously maximise generality, realism and precision'. The argument that Levins proposes is that it is possible to maximise any two of the three, but only at the expense of the third. IFIM maximises generality and precision at the expense of reality. To include additional factors in the model would increase the amount of realism, but this may be at the expense of generality and precision. However, if one of the main purposes of river restoration and rehabilitation schemes is to increase the number of fish present, then a predictive tool that can describe fish requirements in detail is needed. IFIM, as it stands, is a good start but until it takes account of all of the factors that may be relevant it is not sufficiently rigorous. As it stands, millions of pounds may be spent on a restoration scheme based largely on IFIM predictions which do not account for important factors which affect fish habitat. The future of habitat modelling must be to improve upon IFIM to the point where accurate predictions of fish biomass following a restoration can be made, or to abandon IFIM and develop new techniques (such as energistic modelling) that are able to do so. Some of the research requirements that are needed to develop IFIM are detailed in the conclusions chapter in this thesis.

Three dimensional modelling offers more accurate prediction of flow velocities that one dimensional modelling. This is particularly true where flow patterns are more complex and secondary currents are significantly large. As far as habitat modelling goes, the predicted velocities from a three dimensional model offer the possibility to describe the spatial detail of the aquatic environment accurately. However, there are two large drawbacks in the use of three dimensional models in assessing aquatic habitat. The first problem is the lack of a user friendly 3D code which can be easily applied to rivers. The second is that the existing state of knowledge of aquatic species and communities is not yet sufficient to take advantage of the possibilities presented by the predictive powers of 3D flow models. In the literature review of habitat modelling, it was detailed how the importance of the habitat mosaic is beginning to be realised. The existence of a range of habitat types in close proximity is necessary for a good habitat, and not just a particular set of depths and velocities. Unfortunately, the importance of the habitat mosaic to each species is not yet fully understood. The ability to carry out a quantitative assessment of a particular habitat mosaic, using two or three dimensional hydraulic modelling to predict the spatial detail of the flow velocities, is probably a long way off. However, this should be one of the final aims of IFIM.

With the current limitations in the use of IFIM, and in particular with the uncertainties that lie in transferring preference curves derived on one watercourse to any other, the application of the technique is questionable in a lot of cases. The method is able to predict an area of fish habitat, but the results generated can be quite variable and anyway bear no direct correlation to fish biomass. For small improvement schemes, where extensive fish surveys to develop preference curves for the watercourse in question are unlikely to be economically viable, some form of RCHARC type analysis is probably best. Here the flow variability (in terms of velocities and depths) is what is important. A rehabilitation scheme should seek to increase the amount of flow variability so as best to approximate what the natural condition in the river would be. The natural condition can either be taken from reaches on the same watercourse that have not been channelized, or nearby rivers. This type of analysis holds another advantage in that it is not aimed at specific target species, but merely seeks to return the river to as close to a natural condition as possible. Essentially the principle behind this approach is that a river will be colonised by species which are adapted to the type of habitat present. It avoids one of the big problems of IFIM in selecting target species for study. It is possible to select inappropriate species to target a habitat improvement scheme at in IFIM, and hence try to create a fish habitat which will not be in balance with the natural processes in the river. For example, it would not be possible to turn the River Idle into a prolific trout stream without creating structures and flows in the river totally out of balance with the natural process. Such features would be unsustainable in the long term.

For large scale expensive river improvement schemes, where time and money can be invested in developing site specific preference curves for a number of target species, IFIM studies are worthwhile. With input from an experienced ecologist, so that appropriate species are selected for study and realistic assumptions are used in the analysis, IFIM modelling can be used in determining an optimal solution and to produce a form of quantitative measure of habitat improvement.

In the analysis presented here, information on fish preference curves was taken from published texts and only a small number of target species and life stages were selected.

As a result, the results are both lacking in depth and relatively unreliable in terms of providing a meaningful quantitative measure of habitat improvement. In the case analysed, determining a measure of the increase in flow variability following the scheme would probably give as good an indication of habitat improvement, and would be more easy to carry out. However, the approach outlined does have the benefit of differentiating between the effects the rehabilitation scheme has on different species and life stages. In addition, it is considered that it is the principles behind the techniques that have been demonstrated here that are worthy of further consideration. It has been shown that the techniques underlying IFIM can be successfully applied to analyse the results from one and three dimensional hydraulic models to give a quantitative representation of the improvement in available habitat area. With more research into fish ecology, so that all of the relevant parameters can be incorporated into a habitat model, future use of IFIM in conjunction with spatially explicit two or three dimensional models appears to present a potentially useful tool in justifying rehabilitation schemes, on the basis of reliable forecasts of improvements in fish biomass.

5.14. Concluding Comments

The field of habitat modelling is expanding rapidly. A growing body of literature is becoming available on a variety of approaches to habitat studies, new habitat models, and criticisms of existing techniques. It is difficult to predict how far habitat models can evolve. At present the predictive capabilities of hydraulic models are outrunning the available ecological information. As a result, the most urgently required research would be into the complex biology of habitats, in order to be able to make full use of the possibilities offered by spatially explicit habitat mapping using two and three dimensional models.

To conclude, it should be pointed out that recently there has been rapid growth in the technologies available for the surveying of channel morphology, apparent roughness and vegetation distribution. These new advancements can greatly increase the amount and quality of data that can be obtained for the creation of habitat models. These new technologies offer the possibilities for great improvements in the quality and accuracy of results from habitat studies. Remote sensing technologies using GPS and hydro-acoustic arrays can map the subsurface domain to an accuracy of 0.15cm in x-y resolutions, and 15-30cm in elevations (Hardy, 1996). For medium to large rivers, data acquisition at 1-3 metres can still result in several kilometres of surveyed data per day.

Chapter 6

River Idle Sediment Modelling

6.1. Introduction

Accurate prediction of the movement of sediment is of vital importance to civil engineers when considering a wide range of problems. The deposition of sediments can reduce the capacity of reservoirs and interfere with harbour operation. Erosion and scour are major problems in the stability of bridge piers.

For river rehabilitation, it is crucial that designs take into account the movement of sediment or they can quickly be eroded away or covered in deposited material. Alternatively, they may cause adverse amounts of erosion or deposition to occur in other parts of the channel by altering the flow pattern. These effects are explored in more detail in chapter 2. In other words, rehabilitation measures need to be in harmony with the natural movement of material in the river. It should be pointed out that erosion and deposition processes are an inherent feature in the long term pattern of evolution of a river. Geological evolution of a watercourse creates new features in the river and erodes away old ones.

The problem, for the designer of a rehabilitation scheme, is to determine what will be effective in creating habitat and promoting recovery, without causing excessive instability. In the case of the Idle, too much recovery (in the form of bank erosion) would have been undesirable as it might eventually compromise flood defence requirements.

Ultimately, the best way to model the effect of a rehabilitation design would be to use a model that solves sediment transport equations in all three dimensions and predicts erosion and deposition on the bed and banks of the channel. If such a model could also model slope stability, and the mass failure of parts of the bank by undercutting, and it could be simulated for the whole period following rehabilitation with the correct sediment and discharge hydrographs then the evolution of the channel could be modelled in detail. Such a model would be extremely complex and does not, currently, exist (Mosselman, 1995).

With the absence of a wholly reliable predictive model to evaluate channel change, a different approach is required. It is at this point that geographers and civil engineers tend to adopt differing methods. Geographers tend to focus on the wider view of the governing processes and the river as a whole, whilst civil engineers have traditionally focused their attention on a detailed evaluation of specific reaches of the river under consideration. The work presented in this chapter consists of detailed modelling of the movement of sediments at a small number of sites on the River Idle. Thus, it is work carried out from the civil engineering perspective of channel stability. Despite this, the importance of assessing channel stability from the geographical perspective is not underestimated. The assessment of channel stability from the geography perspective is called a geomorphological survey.

The purpose of a geomorphological survey of the river is to identify the dominant processes acting within the river. It should also identify any existing instability (Simon, 1992). To do this may involve the analysis of thousands of kilometres of river channel. Recent advances mean that site evaluation can be coupled with GIS and GPS technology to rapidly synthesise large amounts of data (Simon and Downs, 1995). If the river exhibits signs of instability then the cause needs to be identified (both spatially and temporally), and the stage that the river is at in responding to imposed changes. A geomorphological reconnaissance survey is also useful in identifying conservation value in a river, and identifying those sites which are most suitable for restoration. The survey needs to be carried out by a qualified geomorphologist.

The result of a gemorphological survey in effect provides an overview of the condition of the river. It is critical to know whether the river is stable, or quasi-stable, or is in a state of change. A highly unstable river is unlikely to provide a good location for a rehabilitation scheme as the river is still altering in response to an earlier change. Input from a qualified geomorphologist should also assist in determining what is likely to constitute a stable rehabilitation design. The geomorphologist should make reference to the concept of stream power discussed in chapter 2. Rivers with high stream power will react more markedly to rehabilitation designs than rivers with correspondingly lower stream power. As a result, low energy rivers offer greater possibility for the creation of habitat features as they are less likely to be destroyed by river processes. In a higher energy river much more emphasis must be placed on creating features that are in accordance with the natural tendencies of the river. Another crucial factor is if the river is bedload transporting. Rivers that transport bedload are much more likely to destroy designs which are not in harmony with river processes than non bedload transporting rivers.

In determining what is likely to constitute a stable rehabilitation design, the geomorphologist may make reference to the rational and regime equations that were discussed in chapter 2. These should give some indication of what should constitute stable channel geometry and planform.

The geomorphological survey, in effect, provides the context in which the rehabilitation design is to be set, or the larger picture of the state of the river. Its purpose, as discussed, is to detect any existing instability, evaluate the dominant processes in the river, and to identify those reaches that would benefit most from restoration. The survey should also provide overall guidelines for the design of the rehabilitation measures, such that the scheme is constructed in accordance with the natural tendencies of the river. This may include details such as: the spacing of deflectors and/or riffle sites, the siting of rehabilitation measures in general terms and appropriate sizes for the various features of the design.

Following on from the geomorphological survey, more detailed modelling is required. The work presented later in this chapter provides detailed analysis of likely areas of erosion and deposition at a number of deflector sites on the Idle. Before outlining the detail of this work, it is first necessary to provide a background review of the topic. Thus, the first part of this chapter begins with a description of the 'threshold of motion' whereby sediment transport is initiated on the bed of a watercourse. Following this, there is a discussion of the concept of a critical bed shear stress for sediment transport to commence, and how this value may be determined. The methods for determining the actual bed shear stress are outlined. These can be compared with the critical shear stress to determine if any transport will take place at all. Finally, there is some detail given of several of the most widely used sediment transport formulae.

The detailed modelling of the deflector sites on the River Idle, presented in sections 6.6 onwards in this chapter, can be used to identify the effect that they will have on the localised pattern of sediment erosion and deposition. Results from ISIS Sediment (a module of the ISIS program), and three dimensional results which derive from SHEAR, a FORTRAN program which was written by the author for the purposes of this thesis, are presented. SHEAR processes the results from CFX. Both ISIS and CFX are discussed in chapter 3.

ISIS Sediment was used in this thesis to determine whether the one dimensional program was able to predict erosion at the location of the deflectors. ISIS Sediment requires a sediment hydrograph, as an upstream boundary condition, so that it can predict amounts of erosion and deposition at each cross section. Unfortunately, despite the fact that the bed composition in the Idle was known, no information on sediment discharge rates was available. As a result, a synthetic sediment hydrograph had to be used. This was supplied by the School of Geography. The derivation of the inflow sediment hydrograph was calculated from sediment transport formulae (Brownlie, 1981). The formulae are detailed in the literature review in the early part of this chapter.

Very large quantities of homogenous sediment are known to exist on the bed of the Idle, so the amount of sediment transport is not supply limited. As a result, the use of a synthetic sediment hydrograph becomes less important as the predicted rates of transport are achievable. The results from ISIS Sediment can be used to gain an appreciation of the relative amounts of erosion and deposition along the reaches in question. However, the quantitative amounts of bed change are not reliable.

The FORTRAN program SHEAR processes the three dimensional velocities predicted by CFX. The results provide a spatial representation of the pattern of bed shear stresses. Simulations at different discharges, and pre and post deflectors, provide a good picture of the effect of the deflectors on the bed shear stress. The program is able to predict areas in which the calculated shear stress is sufficient to cause erosion of the substrate. Thus, a visual representation of the effect of the deflectors on bed erosion is created. Areas which are likely to be affected by deposition can be implied by the fact that the installation of the deflectors causes a significant decrease in the bed shear stress. Together, the two programs supply a detailed picture of the effect of the deflectors on the erosion and deposition of sediments, within the overall context of the processes within the river provided by the geomorphological survey. If the techniques outlined here were used at the design stage, rather than after the scheme has been constructed, they could assist in choosing between alternative proposals on the basis of the predicted effects they have on sediment movement. A qualitative comparison of the predictions from ISIS Sediment and SHEAR with the initial measurements of bed elevation change on the Idle following installation of the deflectors, is included towards the end of the chapter.

The study of sediment transport modelling can be highly complex. An extremely large amount of research material has been published on this subject. The discussion and analysis presented here is deliberately kept to what are considered to be the essential features of the topic necessary for this thesis. The work outlined does not deal with some of the complexities of the more in depth models and the detailed approaches which have been applied in other cases. The reason for this is that this thesis deals with several different fields including flow modelling and habitat assessment as well as sediment modelling. Each of these topics could easily provide sufficient work for a study of the length detailed in this thesis. However, as this thesis requires an assessment of all three areas and an analysis of the combined results from each subject, to deal in the detailed complexities of every topic is not a viable alternative.

6.2. Threshold of motion

The study of the erosion of particles of sediment from a river bed is not straightforward. The particles can consist of any number of irregular shapes, and a variety of different sizes. In addition, the bed can be undulating and the particles can interlock with each other to produce an armouring effect counter to the forces tending to move them. The application of a force to the particles will become sufficient to cause movement when it is large enough to overcome the forces tending to resist motion. At the boundary between the riverbed and the water, the passage of the flow causes a shear force to be applied to the exposed particles. Previous research indicates that as the shear force increases from zero a point is reached whereby particle movement begins to take place at a number of points over the bed. Any further small increase in shear stress, above this level, is generally sufficient to initiate widespread sediment movement. At the level of shear stress described, the situation is known as the 'threshold of motion'.

Determining the point at which the threshold of motion exists is only the first problem in analysing sediment transport problems. When movement is initiated and the substrate material consists of sand (as in the case of the Idle) ripples usually form on the bed. These then grow into larger dunes. At moderate Froude Numbers the dunes tend to migrate downstream as sand rolls up the dune crests and is deposited on the leeward side. When the flow is sufficient to bring about a suspended load, the dunes are often washed away. The manifestation of these bed forms makes the accurate prediction of transport rates extremely difficult. They can also significantly effect the water levels and velocities in the river as they have a major effect on the magnitude of the bed roughness. The value of the Darcy Weisbach friction coefficient has been found to vary by a factor of ten as the different bed forms are generated and destroyed (Raudkivi, 1989). The water levels and velocities in turn influence sediment transport properties, and the prevailing bed forms (be they ripples or dunes). A feedback loop operates between the type of bed forms, water levels and velocities and the amount of sediment transport. This makes prediction of sediment transport extremely complex. The literature review and modelling work outlined here focuses on the determination of the threshold of motion, and the use of empirical sediment transport formulae.

6.3. Critical Shear Stress

In 1936, A. Shields carried out some research into the threshold of motion. He used the dimensionless relationship of equation 6-1 and plotted the left hand side of the equation (known as the entrainment function, F_s) against an expression for the Reynolds number. The expression that he used was based on the conditions at the grain surfaces rather than within the main part of the flow, hence equation 6-2:

$$\frac{\tau_{CR}}{(\rho_s - \rho)gD} \propto \frac{\pi A_p}{6} \tan\phi$$

(6-1)

Where τ_{CR} is the critical shear stress, ρ_s is the particle density, D is the particle diameter, A_p is the area of grains divided by the total area, and ϕ is the friction angle of the sediment.

$$\mathrm{Re}_* = \frac{\rho u_* D}{\mu}$$

(6-2)

Where Re* is the Reynolds number relative to the sediment particles, u_* is the shear velocity and μ is the absolute viscosity

Shields' plot of F_s against Re* showed that the type of sediment transport that took place was a function of the position on the diagram. Later, further work was carried out by Rousse. He proposed a line on the Shield's diagram that constituted the threshold of movement. Beyond a certain value of Re* the value of F_s was approximately constant at 0.056. Initially the critical value was thought to be 400, but more recent work places the value between 70 and 150. The Shields' diagram is shown in figure 6.1.Taking F_s=0.056 and substituting into equation 6-1 and rearranging gives equation 6-3.

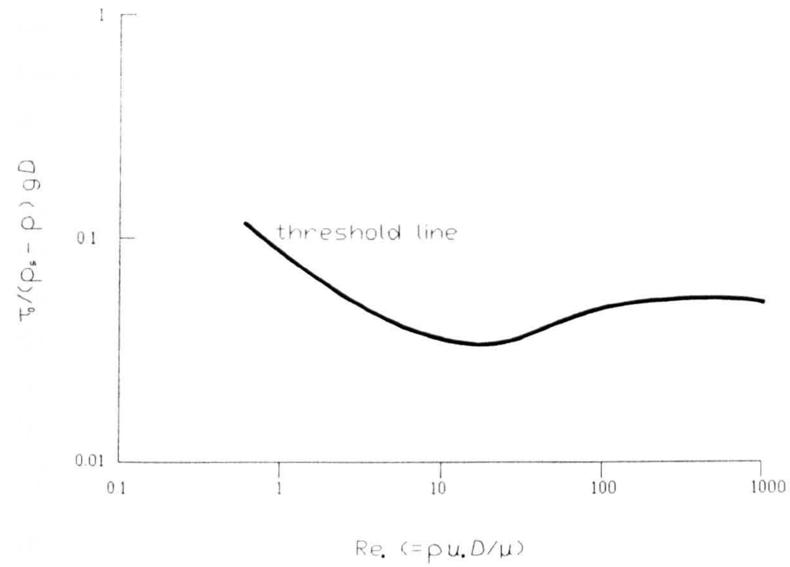


Figure 6.1. The Shields' diagram.

$$\tau_{CR} = 0.56(\rho_s - \rho)gD$$

(6-3)

Equation 6-3 provides a means of determining whether sediment transport takes place, provided that the bed shear stress can be determined. If the calculated value of bed shear stress is greater than the critical value determined from Equation 6-3 then bed erosion, and sediment transport, will take place. If the bed shear stress is equal to or less than the critical value, no transport will take place.

6.3.1 Criticisms of the work of Shields

The work carried out by Shields is of great use in determining whether sediment transport takes place. The equation that derives from it is straightforward to apply and will be used in the analysis of the erosion of sediments in the three dimensional modelling carried out in this thesis. There exists, however, one major shortcoming implicit in the application of Shields' work that is noted here.

The main criticism of the work carried out by Shields is that it is based on sediments of only one grain size (Kuhnle, 1993). Nearly all sediments that occur on the bed of a natural watercourse are composed of particles of a variety of grain sizes. As a result, some later researchers have carried out further work on the principles of the Shields' diagram to see if it can be applied to a sediment containing a mixture of grain sizes. A number of researchers have suggested the use of an index grain size (based on mean or median size). However, it has subsequently been found that it is not only the mean sediment size that is instrumental in determining the critical shear stress, but the make up of the sediment mixture is also important (Wilcock, 1993). For sediment mixtures composed mainly of particles of one diameter (unimodal) the critical shear stress is a function of the mean sediment diameter only. For sediments dominated by two particle diameters (bimodal), the sizes of the particles that contributed to the mixture are also important. Wilcock suggests that a parameter be included in critical shear stress calculations that characterises the degree of bimodality of the sediment composition.

In the case of the River Idle, the composition of the bed material is very unimodal, with approximately 75% of the sampled bed material found to consist of medium sand. As a result, the use of an unaltered form of the Shield's criteria is deemed to be acceptable as the existence of large quantities of homogenous material should dominate the processes of sediment movement.

6.4. Estimating bed shear stress

In order to determine if sediment transport takes place, it is necessary to determine the shear stress at the bed. There are a number of alternative ways to do this. Probably the easiest approach is to assume that uniform flow conditions exist and use a variation on the Chezy equation to calculate the shear velocity, equation 6-4. Bed shear stress can then be determined from the shear velocity using equation 6-5.

$$u_* = \sqrt{gRS_o}$$

(6-4) Where R is the hydraulic radius, and S_o is the bed slope

$$\tau_w = u_*^2 \rho$$

(6-5) Where τ_w is the shear stress at the bed

Another method of determining the bed shear stress is through the direct measurement of the turbulence using sophisticated high frequency flow measurement instrumentation (Graf and Song, 1995). This enables the Reynolds shear stresses (discussed in the review of three dimensional modelling) to be determined and extrapolated down to the bed. The turbulent shear is thus obtained as the effect of the viscous shear is ignored in this case, equation 6-6.

$$\tau_w = \rho u_{*2}^{2} = \left(\mu \frac{d\bar{u}}{dy} - \rho \overline{u'v'} \right)_{y \to 0} = \left(- \rho \overline{u'v'} \right)_{y \to 0}$$

(6-6)

Graf and Song also mention a method for direct measurement of the bed shear stress using a device known as a skin-friction probe. This is a hot film anemometer which is mounted flat on the bed. Results obtained from the probe are said to be unreliable in some circumstances, and the method is not given any more consideration here.

6.4.1 Methods using the velocity profile

Among the most popular methods to determine the shear stress at the bed, are those that use a measured velocity profile. In this approach, the velocity is measured at a number of points vertically above each other. This produces a vertical velocity distribution which is fitted to an equation. The equations that are used to describe the vertical velocity profile all contain the shear velocity, which is related to the bed shear stress. Thus, by rearranging the equation and solving for the observed profile, it is possible to calculate the bed shear stress. This is the method used in the SHEAR program used later in this chapter.

A number of equations have been developed and validated for the purpose of describing vertical velocity profiles. Each one seeks to describe part or all of the velocity profile. Before introducing these equations it is necessary to give some explanation to the existence of various regions that exist within the vertical plane in the flow.

When considering turbulent fluid flow in a watercourse, in the vertical plane, the flow can be subdivided into an inner and outer region. The inner region is directly influenced by the effect of the wall and the mean flow velocities are generally controlled by the wall shear stress, wall roughness, distance from the wall, density, and viscosity of the fluid. The inner, or wall, region may be further subdivided into three parts: a very thin region next to the wall known as the viscous sublayer (where the flow is laminar), a transition region above this where the flow changes from laminar to turbulent, and then the turbulent part of the inner region (also known as the log law layer). The flow in the outer region is fully turbulent . Velocities in the outer region are only indirectly affected by the wall, and are more heavily influenced by turbulent shearing (Kirkgoz, 1989). The three main constituent parts of a vertical velocity profile (viscous sublayer, log law and outer regions) are shown in figure 6.2. The diagram implies that the point at which the velocity is zero does not necessarily lie at the top of the sediment particles on the bed. This matter is discussed in more detail later in this chapter when the SHEAR program is introduced.

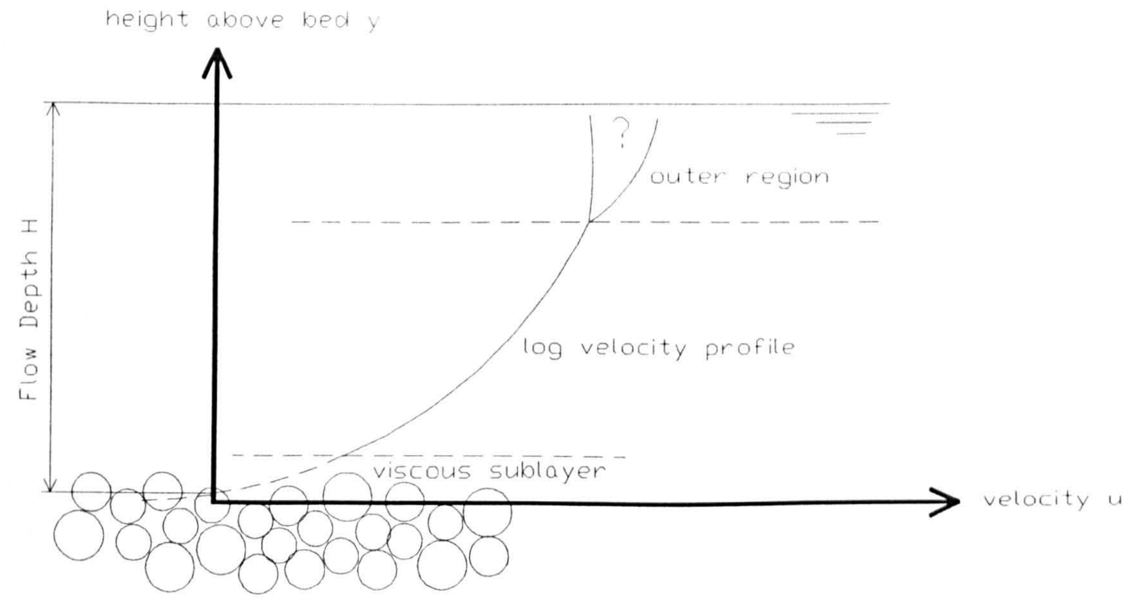


Figure 6.2 The regions in a vertical velocity profile. Adapted from Smart, 1999.

In the viscous sublayer, the total shear stress is constant at any distance from the wall and equal to the wall shear stress, τ_w. The velocity distribution is given by equation 6-7.

$$\frac{u}{u_*} = \frac{u_* y}{\nu}$$

(6-7) Where y is the distance up from the boundary, and ν is the kinematic viscosity

In the fully turbulent part of the inner region, the velocity distribution is given by von Karman-Prandtl's (von Karman 1930, Prandtl 1932) "law of the wall", equation 6-8.

$$\frac{u}{u_*} = A \ln \frac{u_* y}{\nu} + C$$

(6- 8) Where A and C are constants which commonly take the values: A=2.5 and C=5.5

In the outer region of flow the velocity distribution can be given by a modified form of the law of the wall, which incorporates some extra terms known as the modified wake function. Alternatively, a new equation known as the velocity defect law, equation 6-9 (Vedula and Achanta, 1985) can be used.

$$\frac{u_m - u}{u_*} = -2.5 \ln \frac{y}{\sigma} + 2.5$$

(6-9) Where u_m is the maximum velocity in the distribution, and σ is the thickness of the boundary layer

The above three formulae are discussed in terms of dimensionless coefficients used in specifying the boundary roughness and 'wall functions' in the review of three dimensional modelling. They are also quoted here in a slightly different form for completeness.

As well as the above three formulae, that fit specific regions in the velocity profile, two more equations can be used. The choice of which of these two equations to use is dependant on whether the flow is hydraulically 'rough' or 'smooth'. In hydraulically rough flow $u_{*}D/u > 70$ and the sediment particles protrude out of the laminar sublayer. In this case viscosity effects are less important. In hydraulically smooth channels, $u_{*}D/u < 5$ and the sediment is completely immersed in the laminar sublayer. Viscous effects are important. Note that the limit of the laminar sublayer is given by $u_{*}y/u < 5$. There then follows the transition region up to the fully turbulent core at $u_{*}y/u > 70$. The formula for the velocity profile in hydraulically smooth flow is given by equation 6-10 (Kabir and Torfs, 1992).

$$\frac{u}{u_{*}} = 2.5 \ln \frac{y}{y_o} + 8.5$$

(6-10) Where y_o is the point where the prolonged curve of the velocity distribution intersects the line of zero velocity. This is usually taken as being equal to $\sigma/117$.

In hydraulically rough flow, the velocity profile is given by equation 6-11.

$$\frac{u}{u_{*}} = 2.5 \ln \frac{y}{k_s} + 8.5$$

(6-11) Where k_s is a measure of the sand grain roughness

One of the main problem lies in determining what portion of the velocity profile is matched to each of the above formulae. Or, in other words, to what height the viscous sublayer and inner and outer regions extend within the flow. There is a wide body of reported evidence on this topic, but no firm consensus appears to have emerged.

In practice, the thickness of the laminar sublayer is often so small that unless velocities are measured extremely close to the bed all of the recorded data will lie outside of this region. Therefore the main problem occurs in determining the heights of the inner and outer regions, and in checking the validity of the stated equations within these regions. The velocity distribution in the inner region is best represented by one of the log law based relations, equations 6-8, 6-10 and 6-11. The formulae in the outer region is best represented by a log law relation with extra terms added, or alternatively the velocity defect law of equation 6-9.

Versteeg and Malalasekera (1995) recommends that the log-law (inner) region be taken as lying between the first 2% – 20% of the thickness of the boundary layer. In an open channel flow the boundary layer has full opportunity to develop so the top would be expected to be at or near the water surface. Bridge and Jarvis (1977, 1982) found that the first 15% of the boundary layer was logarithmic, whilst in another study the first 20% was found to be suitable (Bridge and Jarvis, 1976). Ferguson et al (1989) and Ferguson et al (1992) found that at least 50% of the boundary layer fitted a logarithmic expression. Cardoso et al (1989) found that the log law described their data well over the entire channel depth. In direct contrast, Biron et al (1998) demonstrated, using laboratory flume results, that use of the whole velocity profile can produce an underestimation of the bed shear stress. Nezu and Rodi (1986) showed that for a smooth bed channel the outer region occupied as much as 80% of the flow depth.

A review of much of the available research suggests that the most widely adopted approach for the determination of bed shear stresses using a fitted velocity profile, is to apply a log-law based formula to the bottom 20% of the velocity profile. This is the method employed by Kabir and Torfs (1992). The remaining part of the velocity profile can then omitted from the analysis and is taken as lying either in the outer region of the boundary layer, or outside of the boundary layer altogether. This is the method employed in the SHEAR program, which is outlined later in this chapter. In this program the formula for hydraulically rough and smooth flow, as well as the law of the wall, are all used in the determination of the bed shear stress, using the bottom 20% of the velocity profile. In this case, a three dimensional flow model was used to predict the vertical velocity profile at each point in the solution domain.

6.5. Sediment Transport

If the bed shear stress exceeds the critical shear stress determined from the Shields' diagram, then sediment transport will take place. The process by which bed erosion and transport is initiated is not straightforward. Firstly 'ripples' form on the bed. Sediment accumulates on these ripples until they become sufficiently large to be classified as 'dunes'. In flows with relatively low Froude Numbers the dunes migrate downstream as sediment particles roll over the dune crests and are deposited on the leeward side. This is bed load transport.

If the flow is sufficient, particles become entrained in the current to form a suspended load. The dunes can be washed away. Current thinking suggests that a relationship exists between the parameters of sediment transport (sediment properties, shear stress, dune size, transport rate, Froude Number, fluid properties, bed roughness). However, the precise form of this relationship has yet to be determined.

An explanation for what is occurring during sediment transport can be found by considering in more detail the mechanisms which are present. Particles on the channel bed exist in the laminar sublayer where the flow is slow moving or even stationary. However, the sublayer is not stable as eddies from the turbulent main body of the flow, and possessing high momentum, occasionally penetrate it and eject some of the low momentum fluid. The momentum difference between the sublayer and the main body of the fluid also generates a shearing action which produces more eddies. Individual grains are, therefore, subject to a fluctuating force tending to dislodge them from their position. If the force becomes sufficient they will begin to roll over neighbouring grains. If movement becomes widespread the pattern of forces will become chaotic as grains collide with each other, and with stationary grains on the bed. As the bed shear stress increases, movement penetrates deeper into the granular material on the bed.

For particles of sediment to be drawn into suspension, an upward force must be applied to lift them up into the main part of the flow. The fluctuating components of the turbulent flow provide the force on the particle to do this. Flow separation on the top of the particle also provides an initial lift force to draw the sediment upwards from the bed. If the turbulent motion is sufficiently intense above the laminar sublayer, the eddying action will lift the particles up into the main part of the flow. As would be expected the finer, lighter particles will be most readily lifted into suspension. In practice, all sediment transport occurs either solely as bed load, or as a mixture of bed and suspended load.

6.5.1 Sediment Transport Formulae

As can be inferred from the discussion so far, sediment transport is a complex phenomenon and does not yield readily to any mathematical analysis. A large number of equations have been published for the prediction of sediment transport. Some of the most popular are those published by Du Boys in 1879, Einstein in 1942, Bagnold in 1966 and Engelund and Hansen (1967). DuBoys and Einstein formulae deal with bed transport only, while Bagnold, Brownlie and Engelund and Hansen and Ackers and White are formulae for combined bed and suspended load.

The work presented by DuBoys uses a 'tractive force' idea. The method is based on the fundamental concept that the rate of sediment transport is related to the shear stress, equation 6-12.

$$q_s = f(\tau_W)$$

(6-12) Where q_s is the sediment transport rate

Many researchers have subsequently used this basic assumption to produce their own relationships between the rate of sediment transport and the shear stress. One of the most popular was produced by Shields in 1936, equation 6-13.

$$q_s = \frac{10qS_o\rho^2}{\rho_s}\frac{(\tau_W - \tau_{cr})}{(\rho_s - \rho)^2 gD}$$

(6-13)

Einstein's work in this area is based upon the determination of the statistical likelihood of the occurrence of an eddy capable of transporting a particular grain. The approach also uses the idea of the particles moving in a series of jumps along the bed. Each jump is of a determined length, and a certain number of jumps occur during each time period. Combining the likelihood of a particle being eroded with how far and how often it jumps along the bed, gives rise to a sediment transport formula. The steps in the derivation, and the formula itself are not quoted here. The interested reader can refer to Chadwick and Morfett, 1986.

Similarly, the derivation and formula for the work done by Bagnold on sediment transport are not quoted here. The idea used by Bagnold is based on the concept of a stream power and the immersed weight of sediment particles. Stream power is defined in equation 6-14. For the bed load part of the calculation, the fraction of the stream power deemed to be used to transport sediment is equated with the immersed weight of sediment particles. For the suspended load, an effective upward velocity is taken to exist which counteracts the immersed weight of the particles. The Bagnold formula requires the estimation of a number of coefficients. A certain degree of uncertainty lies in this estimation.

$$P = \rho g u B H S_o$$

(6-14) Where P is the stream power, B is the channel width and H is the flow depth

The total load formula suggested by Brownlie was derived using dimensional analysis and regression fitting of laboratory and field data (Brownlie, 1981). Graded sediments can be represented by the use of the D_{50} bed material size. The formula is given in equation 6-15:

$$\bar{c}_T = 727.6 C_F \left(F_g - F_{gc}\right)^{1.978} S^{0.6601} \left[\frac{R}{D_{50}}\right]^{-0.3301}$$

(6-15) c_T is the sediment concentration in ppm, F_g is the grain Froude number defined by equation 6-16 and F_{gc} is defined by equation 6-21, C_F is 1.0 for laboratory data or 1.268 for field data, S is the water surface slope

$$F_g = \frac{u}{\left(\left(\frac{\rho_s}{\rho} - 1\right) g D_{50}\right)^{0.5}}$$

(6-16) R_g is the grain Reynolds number defined by equation 6-17

$$R_g = \frac{\left(g D_{50}^{\ 3}\right)^{0.5}}{31620 \nu}$$

(6-17)

In order to determine the value of F_{gc} it is first necessary to calculate Y from equation 6-18

$$Y = \left(\sqrt{\frac{\rho_s}{\rho} - 1 R_g} \right)$$

(6-18)

A dimensionless critical shear stress, τ_{ci} is determined from equation 6-19

$$\tau_{ci} = 0.22Y + 0.06(10)^{-7.7Y}$$

(6-19)

A geometrical standard deviation of the particle size is given by equation 6-20

$$\delta = \sqrt{\frac{D_{84}}{D_{16}}}$$

(6-20)

F_{gc}, the critical grain Froude number, can now be determined from equation 6-21

$$F_{gc} = 4.596\tau_{ci}^{0.5293} S^{-0.1405} \delta^{-0.1606}$$

(6-21) S is the water surface slope

For any sediment movement to take place, F_{gc} must be greater than F_g

The formula suggested by Engelund and Hansen was developed by equating the work done by the drag forces of the flow to the potential energy gained by particles as they move up the face of the dune (Engelund and Hansen, 1967). The potential energy required to elevate the particles through a height h_1 is given by equation 6-22. This is related to the drag forces on the moving particles during the same time. Assuming the particle velocity is proportional to u_* and the shear stress transferred to the particles can be taken to be $(\tau_W - \tau_{CR})$ it is possible to write equation 6-23. After some algebra it is possible to arrive at the Engelund-Hansen total load equation shown in equation 6-24.

$$(\gamma_s - \gamma_f)\frac{q_s}{\gamma_s}h_1$$

(6-22) γ_s is the specific weight of the sediment, γ_f is the specific weight of the fluid

$$\frac{q_s}{\gamma_s}(\gamma_s - \gamma_f)H = K(\tau_0 - \tau_{CR})Lu_*$$

(6-23) K is a non dimensional coefficient, H is the flow depth and L is the length of the dune

$$q_s = K\frac{0.05Bu^2H^{1.5}S^{1.5}}{(\gamma_s - 1)^2 D\sqrt{g}}$$

(6-24) W is the flow width, D is the sediment diameter and g is the acceleration due to gravity.

6.5.1.1 Ackers and White

One of the most common formula used in sediment transport is that suggested by Ackers and White in 1973 and subsequently updated in 1993 (Ackers and White, 1973), (Ackers and White, 1993). The original form of the equations was determined by considering the appropriate physical considerations to take into account, and using dimensional analysis. The coefficients in the equations were equated from empirical data. Again, the formulae represent both bed and suspended load.

This work focuses on the use of three dimensionless coefficients, G_{gr}, F_{gr} and D_{gr}. G_{gr} is based on the concept of the stream power. For the bedload, the relevant stream power is related to the flow velocity and the bed shear stress. Conversely, the total stream power is taken to be the relevant parameter in transporting suspended load. F_{gr} is the particle mobility factor and is a function of shear stress and the immersed weight of the particle. The critical value of F_{gr}, where sediment transport commences, is given by A. The factor D_{gr} represents the relationship between the immersed weight of the grains and viscous forces. The most recent versions of the Ackers and White Formula are given in equations 6-25 to 6-32.

$$G_{gr} = \frac{q_s D_m}{qD}\left[\frac{u_*}{u}\right]^n = C\left[\frac{F_{gr}}{A} - 1\right]^m$$

(6-25) Where q is the discharge and D_m is the mean depth

$$F_{gr} = \frac{u_*^n}{\sqrt{gD[(\rho_s / p) - 1]}}\left(\frac{u}{\sqrt{32}\log(10D_m / D)}\right)^{1-n}$$

(6-26)

$$D_{gr} = D\left(\frac{g[(\rho_s / \rho) - 1]}{v^2}\right)^{1/3}$$

(6-27)

For $D_{gr}>60$

$m=1.78$, $C=0.025$, $n=$, $A=0.17$

(6-28)

For $1<D_{gr}<60$

$$m=\frac{6.83}{D_{gr}}+1.67$$

(6-29)

$$C=10^{\left(2.79\log_{10}D_{gr}-0.98\left(\log_{10}D_{gr}\right)^2-3.46\right)}$$

(6-30)

$$n=1-0.56\log D_{gr}$$

(6-31)

$A = 0.14 + 0.23 / \sqrt{D_{gr}}$

(6-32)

The Ackers and White, and Engelund-Hansen formulae are both used in the one dimensional sediment transport modelling, using a module of the ISIS package. This work is described later in this chapter. Both formulae have been calibrated against a wide range of data and the results obtained were good. However, as has been stated, prediction of rates of sediment transport is problematical. The situation is well summarised by the following quote: '*The question regarding which formula to use to predict sediment transport is difficult to answer. All the formulae given are empirical and are based on experimental data. The measurement of sediment transport in the laboratory can be accurate but the experiments are undertaken in straight, rectangular flumes which are not necessarily representative of natural river channels. The measurement of sediment transport in the field is extremely difficult and the accuracy is normally low*' (Fisher, 1995).

6.6. One Dimensional Sediment Modelling of the River Idle

The one dimensional sediment modelling carried out for this thesis uses a module of the ISIS package called ISIS Sediment. The module allows the use of both the Ackers and White and Engelund and Hansen formula. Both were used so that the results from the twin approaches could be compared. Both approaches have been discussed earlier in the chapter. ISIS Sediment can be used to predict sediment transport rates and changes in bed elevation over a certain time period with a specific incoming discharge (or discharge hydrograph).

Flow deflectors are included in the ISIS model by incorporating them into the channel cross sections. Extra sections are added into the ISIS model around the deflectors so that the detail of the localised narrowing of the cross section width, that they produce, can be accurately simulated.

ISIS Sediment works by calculating the hydraulic variables (discharge, depth and velocity) in the normal way. Then, starting from the upstream end of the network, the program calculates the sediment transport capacity and solves the sediment continuity equation to give the change in bed elevation at each section. Finally, the bed elevations at each cross section are updated to account for the change arising out of the erosion or deposition. The sediment continuity equation is given by equation 6-33. ISIS Sediment does not allow for the effects of sediment armoring.

$$(1-\lambda)B\frac{\partial b_e}{\partial t}+\frac{\partial q_s}{\partial x}=0$$

(6-33) λ is the bed porosity, B is the water surface width, b_e is the bed elevation, t is time, q_s is the sediment transport rate (m^3/s) and x is the distance in the flow direction.

ISIS Sediment allows a range of particle sizes to be used to represent the composition of the bed material, together with the percentages with which each diameter is present. Thus it was possible to accurately represent the makeup of the bed material, as had been previously determined by site sampling (carried out by University of Nottingham School of Geography).

6.7. Three Dimensional Sediment Modelling of the River Idle

As with habitat modelling, a FORTRAN program (in this case called SHEAR) was written by the author to process the results generated by CFX. The program SHEAR takes the predicted velocities from CFX and calculates the bed shear stress at every node point in the solution domain. It then determines if the bed shear stress, at each node, is sufficient to cause erosion of the sand substrate. Using the output from SHEAR it is possible to produce contour plots of the bed shear stress pre and post deflectors, at specific discharges. The predicted area where the bed would be eroded pre and post deflectors can also be plotted. The results can be used to draw conclusions about the effect of the deflectors on the pattern of erosion and deposition in the river, within the overall context of the stability of the channel. The purpose of SHEAR is to give a qualitative picture that can be used to analyse the effect of river rehabilitation works.

6.7.1 The SHEAR Program

The SHEAR program contains a value of the typical grain diameter of the substrate. In the case of the River Idle the bed was typically composed of between 65 - 80% medium sand, prior to the installation of the deflectors. Medium sand represents a particle diameter range between 0.25mm and 0.42mm. Using equation 6-3, which is based on the Shields diagram, a critical shear stress for erosion of the bed is calculated based on a grain diameter of the substrate which is typically commensurate with medium sand. Some plots are given below in which the critical shear stress is based on a particle diameter outside of the range 0.25mm to 0.42mm. The reasons for this are discussed in the results section.

The next stage in the program is to calculate the actual bed shear stress at each point in the solution domain. Earlier in the chapter, a number of log law based methods were outlined to describe the vertical velocity distribution in open channel flow. In particular, the law of the wall (equation 6-8), a smooth wall formula (equation 6-10) and rough wall formula (equation 6-11) were outlined, all of which are generally taken to be applicable to the bottom 20% of the velocity profile.

Rearranging the law of the wall gives equation 6-34:

$$u = 2.5u_* \ln y + 2.5u_* \ln \frac{u_*}{v} + const_1$$

(6-34)

Rearranging the smooth wall formula gives equation 6-35:

$$u = 2.5u_* \ln y + 2.5u_* \ln \frac{1}{y_0} + const_2$$

(6-35)

Rearranging the rough wall formula gives equation 6-36:

$$u = 2.5u_* \ln y + 2.5u_* \ln \frac{1}{k_s} + const_3$$

(6-36)

SHEAR uses the vertical velocity distribution to plot values of u against $\ln y$, using the velocities in the bottom 20% of the depth at each node point in the solution domain. The velocities used, at each point, are the resultant of the stream wise and lateral components. Using the method of least squares, a line of best fit is put through the plotted values. The gradient of this line is equal to $2.5u_*$ (as can be seen from the term on the left hand side and the first term on the right hand side in each of the three formulae above). As a result, u_* can be determined at each node point, and this means that the bed shear stresses can also be calculated using equation 6-5. If the calculated bed shear stress, at a given point, is greater than the critical shear stress then erosion of the bed takes place at that point. If the calculated bed shear stress is less than or equal to the critical shear stress no erosion takes place at that point. SHEAR also adds up the area of bed erosion by summing the plan areas of the cells nearest the bed where erosion has been deemed to take place.

The program SHEAR does not include two effects which are significant in evaluating erosion in a real river. Firstly, the program can only be used to calculate the erosion of particles on the bed. For particles that are on the river bank, part of the self weight of the particle would act in aiding to dislodge the particle from the slope. This effect is not included as it would be hard distinguish between locations on the bed and locations on the bank. Also, in reality bank slips would occur when erosion of the toe of the bank is sufficient to increase the bank slope to a value greater than critical. To include this effect would require some form of bank stability analysis, in combination with evaluation of rates of erosion. This is beyond the scope of the analysis presented here.

Another possible future improvement to SHEAR would be to incorporate a statistical check in the program to examine how well the modelled velocities fitted a logarithmic distribution. Using a statistical check, additional points (extending successively further from the bed) could be included in the vertical velocity profile used to calculate the shear velocity on the proviso that they fitted the existing distribution. This would also help to indicate the height to which the logarithmic (inner) region of the boundary layer extended.

Some researchers have pointed out that the level at which the velocity is zero should not necessarily be taken as the top of the granular material that constitutes the bed (Kabir and Torfs, 1992), (Biron et al 1998) (Kirkgoz, 1989). Instead the velocity profile can extend into the bed material layer, and the true level at which the velocity should be taken as zero lies somewhere between the top and bottom of the sediment grains on the bed. This effect is of most concern where the sediment grains are relatively large and the flow is hydraulically rough. These conditions are often met where the bed is composed of gravel, or roughness elements of a larger diameter. In the case of the Idle, the bed material is largely composed of sand and so this effect has been ignored.

In chapter 3 the means by which boundary roughness is incorporated into three dimensional models, and CFX in particular, is explained. This involves modification (based on the effective grain roughness of the boundary material) to two dimensionless coefficients, the log layer constant and the sub layer thickness. These modified values are included in the command file for the CFX simulation. Modifying the values influences the near wall velocity profile in a way that accurately represents the effective grain roughness. With this in mind, the method of determining the wall shear stress used in the SHEAR program would seem to be anomalous. The near wall velocity profile has been partly determined by the modified values incorporated in the command file. The resulting velocity profile has then been used, by SHEAR, to determine the wall shear stress. However, the method used is reasonable as the near wall velocity profile is only partly determined by the values of the dimensionless log layer constant and sub layer thickness that are input to the command file. The turbulence in the near wall region is also a major factor in the determination of the boundary shear stress. This can be seen in equations 3-46 and 3-47 in chapter 3. The influence of the turbulent properties of the flow can also be seen by the fact that direct measurement of the Reynolds stresses can be used to determine the boundary shear stress, as shown by equation 6-6. The situation is summarised by the quote *"A given vertical profile of velocity does not reflect the size of material at the base of the profile but is a consequence of hydraulic conditions extending upstream of the base of the profile."* (Smart, 1999)

The fact that the direct measurements of turbulence properties can be used to determine the wall shear stresses offers an alternative method to the use of the observed vertical velocity profile (as employed by SHEAR). All of the three dimensional simulations carried out in this thesis used the k-ε turbulence model. As a result, the Reynolds stresses are not solved explicitly and so equation 6-6 could not be used to determine the wall shear stress. However, equation 3-46 of chapter 3 directly relates the turbulent kinetic energy k to the wall shear stress. By using the calculated value of k in the cells nearest to the wall, it would be possible to calculate the wall shear stress at each point. Nevertheless, it is considered that the method using the calculated vertical velocity profile, as employed by the SHEAR program, has greater potential flexibility. The velocities at different percentages of the flow depth can be used in the determination of the wall shear stress, as required. Indeed, the program could be used to assess the effect on the predicted bed shear stresses of using velocities from different proportions of the flow depth. This test has not been carried out as part of this thesis, as no site measurements of bed shear stresses are available with which to calibrate the predicted values. The SHEAR program could also be used to analyse bed shear stresses from site measured velocity profiles, assuming sufficient measurements were taken near the bed.

6.8. Results

6.8.1 One Dimensional Results

The results from the ISIS Sediment module are presented in long section form showing the predicted change in bed elevation caused by the erosion or deposition of sediment (figures 6.3 to 6.6). Plots are presented for the whole of section 6 (which incorporates 5 deflectors) and for deflector sites 3c and 3f. Sites 3c and 3f were simulated for one hour and with a constant discharge of 1.204 m^3/s and 1.252m^3/s respectively (the 'initial' flows). Section six, which includes five deflectors, was simulated with a real rainfall event recorded at the Mattersey gauging station which lasted for a little over 62 hours. During this storm the discharge rose from 4 m^3/s to a peak of almost 8 m^3/s before returning to 4 m^3/s. Spikes of erosion are evident at the location of the deflectors in each plot (one for each of the three wing deflectors in 3c and one for each of the two wing deflectors at 3f).

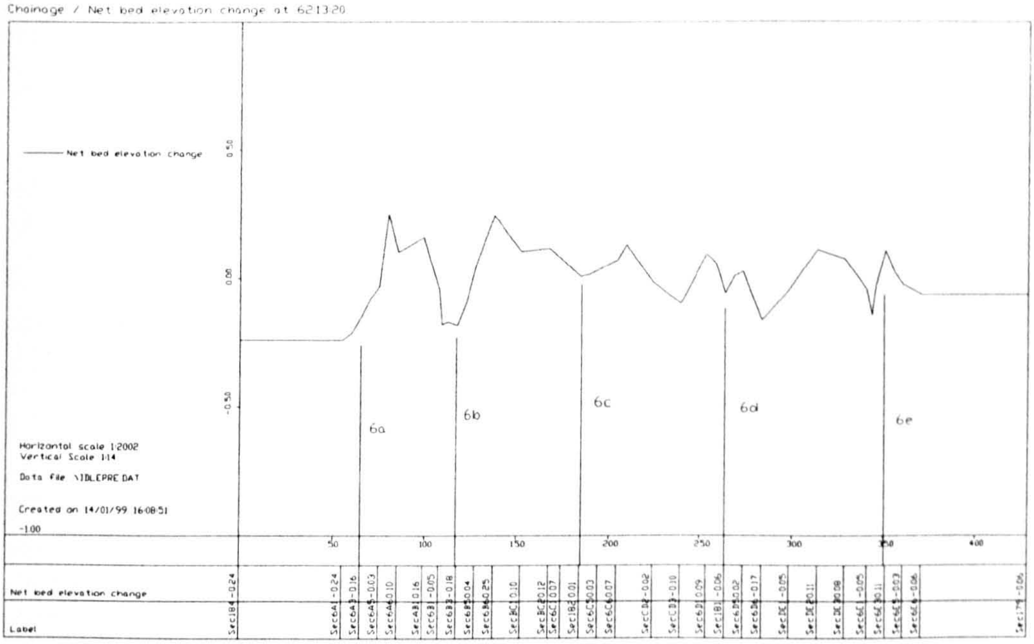


Figure 6.3. Predicted bed change for section 6 pre deflectors. Based on the Ackers and White formulae

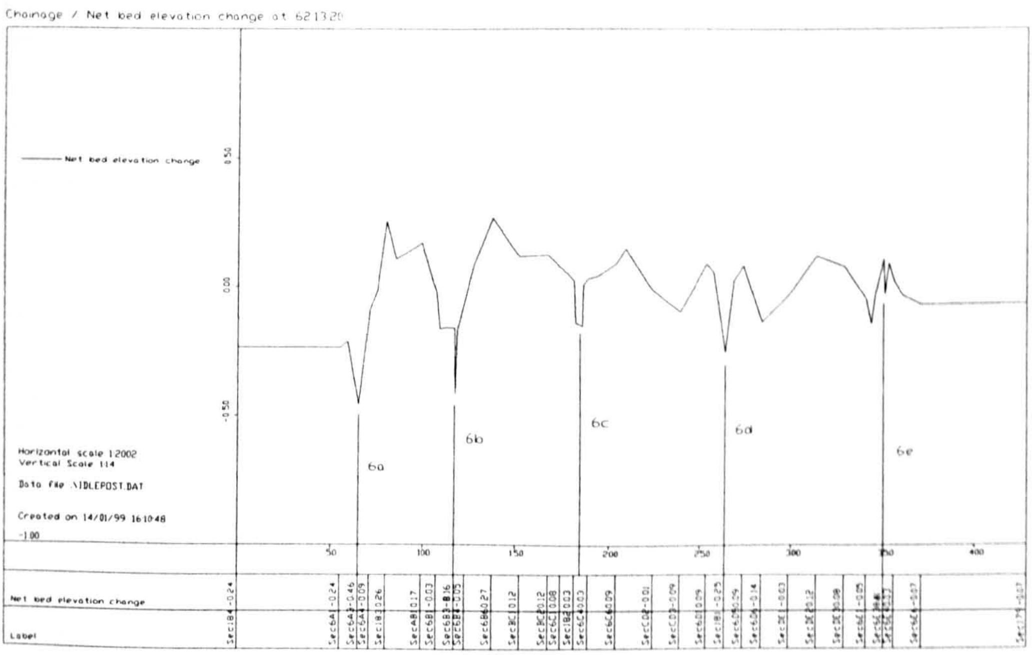


Figure 6.4. Predicted bed change for section 6 post deflectors. Based on the Ackers and White formulae

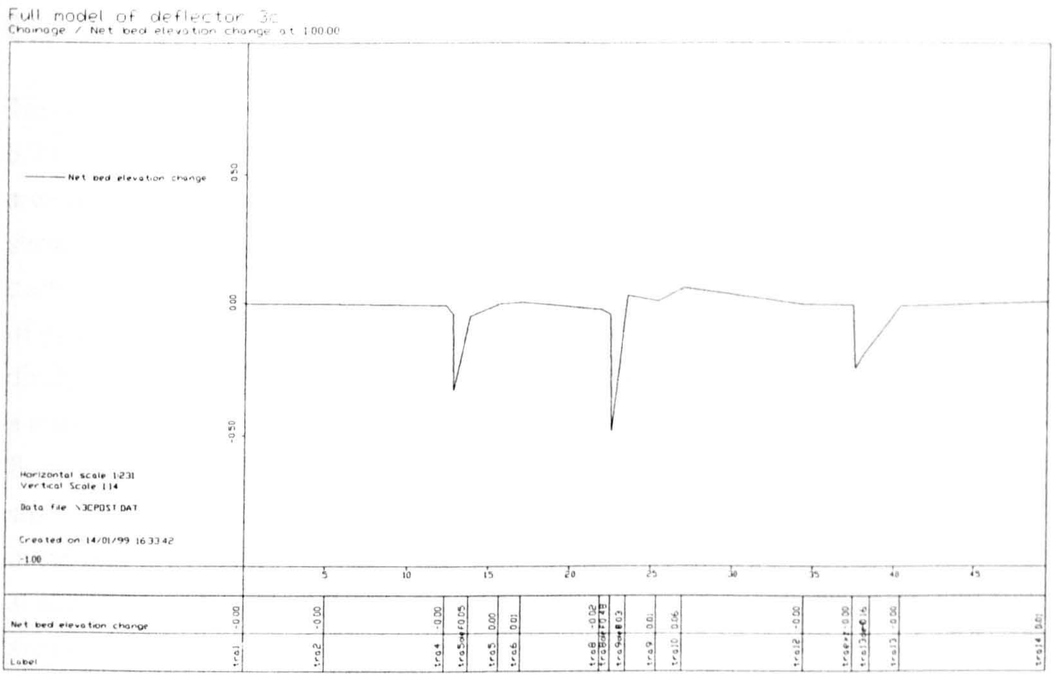


Figure 6.5. Predicted bed change for site 3c post deflector. Based on the Engelund Hansen formulae

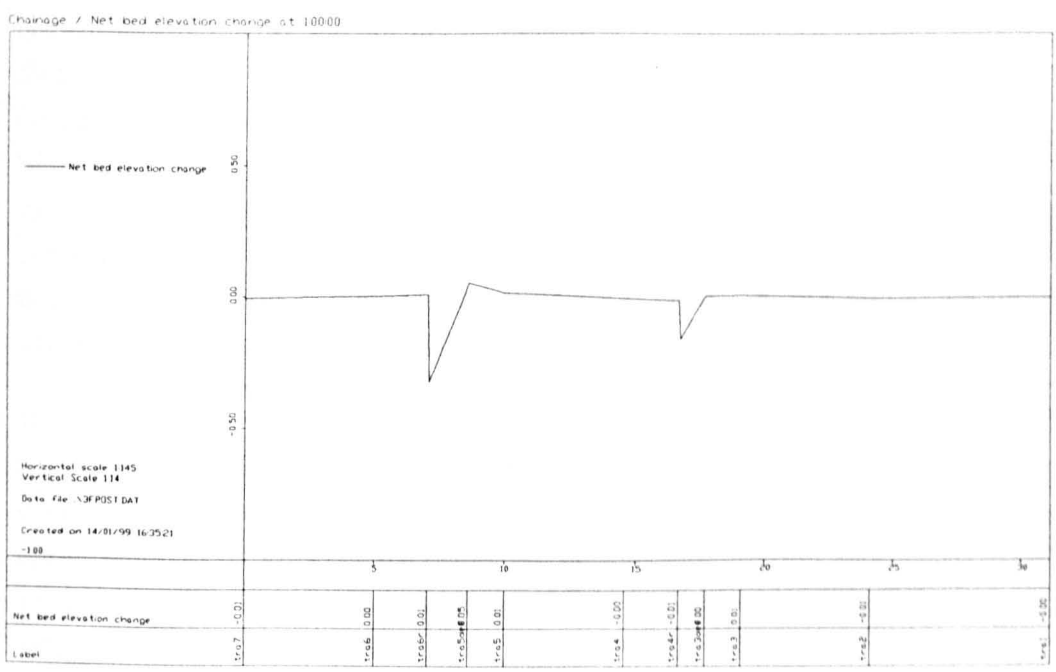


Figure 6.6. Predicted bed change for site 3f post deflector. Based on the Engelund Hansen formulae

6.8.2 Three Dimensional Results

Three dimensional results, produced by the program SHEAR, are presented in figures 6.7 to 6.20 for the three deflector sites on the River Idle. Plots showing the predicted area of the bed that will be eroded, are presented for the 'initial' and 'high' flows. Bed shear stress plots are presented for 'low' flows. Including both types of result plot for each deflector site and at each discharge would have given rise to an excessive number of graphs. By including results in the manner prescribed above, results for different discharges and deflector sites are included while keeping the overall number of plots to a reasonable level. Relatively little erosion of the constituent bed material occurs at low flow, however the plots included at this discharge show the changing pattern of shear stress at the bed caused by the deflectors. Greater areas of erosion are predicted at the initial and high discharges, although the bed shear stresses do not need to be presented at this discharge as the patterns are very similar to those at the low flow. The predicted areas of erosion for each particle diameter are summarised in tables 6.1 to 6.5. The extreme low flow ($0.3m^3/s$) was not included here as it represents a discharge at which the movement of sediment will be negligible.

Medium sand represents a particle diameter range between 0.25mm and 0.42mm. Some of the plots of eroded area of bed use particle diameters above this range. This is useful as it indicates the extent to which the calculated bed shear stress exceeds the critical shear stress for the erosion of medium sand. It should be borne in mind that there are some limitations and assumptions in the approach used here. These are detailed earlier in the review of the work of Shields, and in the description of the SHEAR program given earlier in this chapter. Thus, the results given here should be viewed with these presuppositions in mind. Where the program predicts erosion will occur for a particle diameter greatly in excess of 0.42mm, this gives a good indication that the predicted shear stress will be sufficient to erode the sand bed, at the specified discharge, even bearing in mind the limitations and assumptions in the approach.

In a small number of cases, the eroded area of bed is based on a particle diameter of 0.18mm. Although this represents a particle diameter below that of medium sand, it is considered that the inclusion of these plots is worthwhile as they do contribute to an appreciation of the effect of the deflectors on the pattern of bed shear stress and likely erosion. In addition, a small percentage of finer material does exist on the bed of the Idle, and this may be eroded at a shear stress less than that which is required for the erosion of the medium sand (depending on how the mechanism of sediment armoring detailed earlier in this chapter operates in this case). In all cases where results for a particle diameter of 0.18mm are included, a diameter of 0.25mm is also shown.

In the plots that follow, the areas of the bed of the Idle where the predicted shear stress is insufficient to cause bed erosion are shown in light brown. Areas of the bed where erosion is predicted are shaded in successively darker shades of brown. Each shade darker represents predicted erosion of the next highest particle diameter. For example in results figure 6.16 there are four shades of brown (excluding the background colour which represents no erosion). The lightest shade represents erosion for a particle diameter of 0.25mm and the darkest represents erosion for a diameter of 0.5mm.

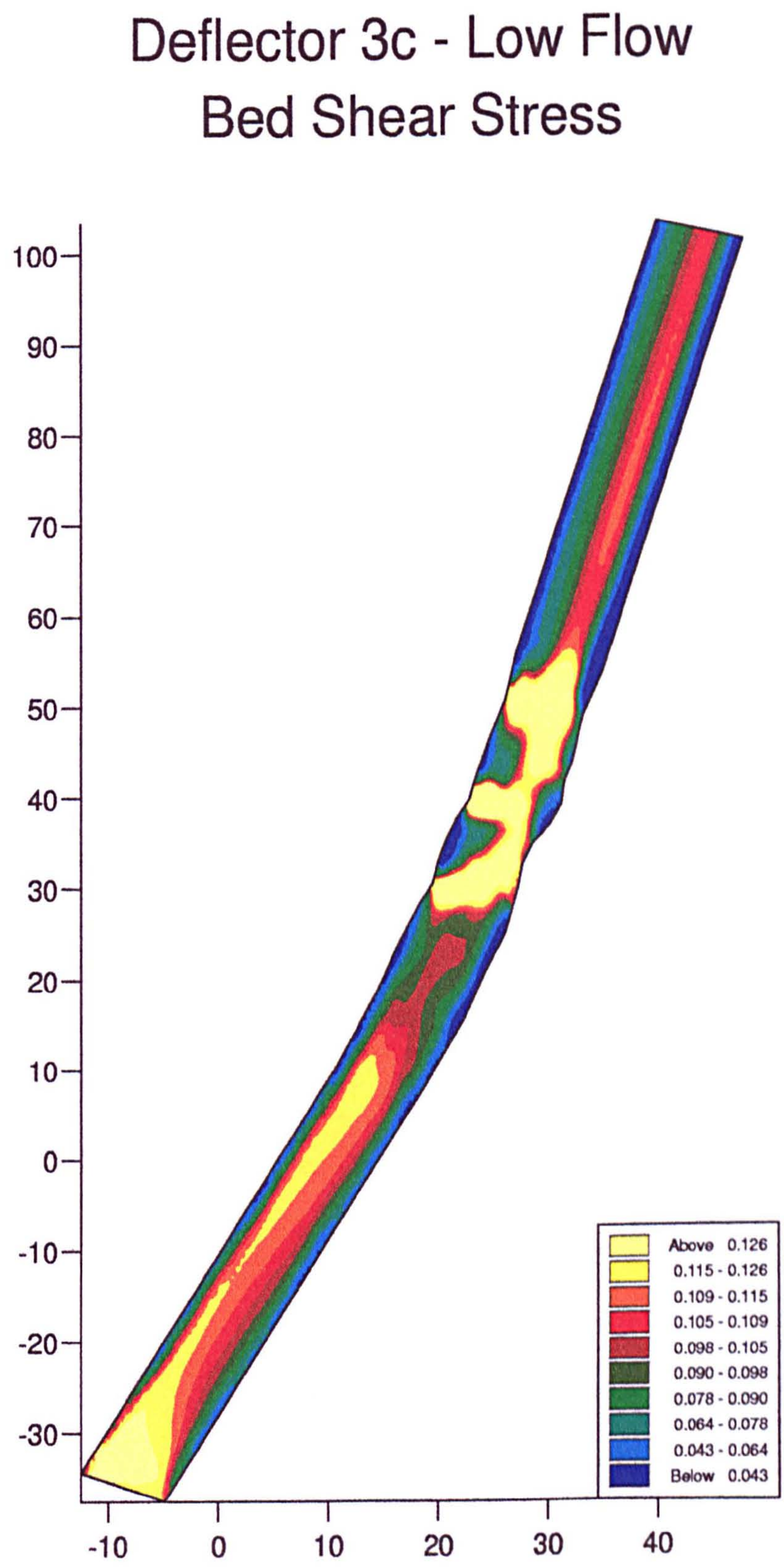
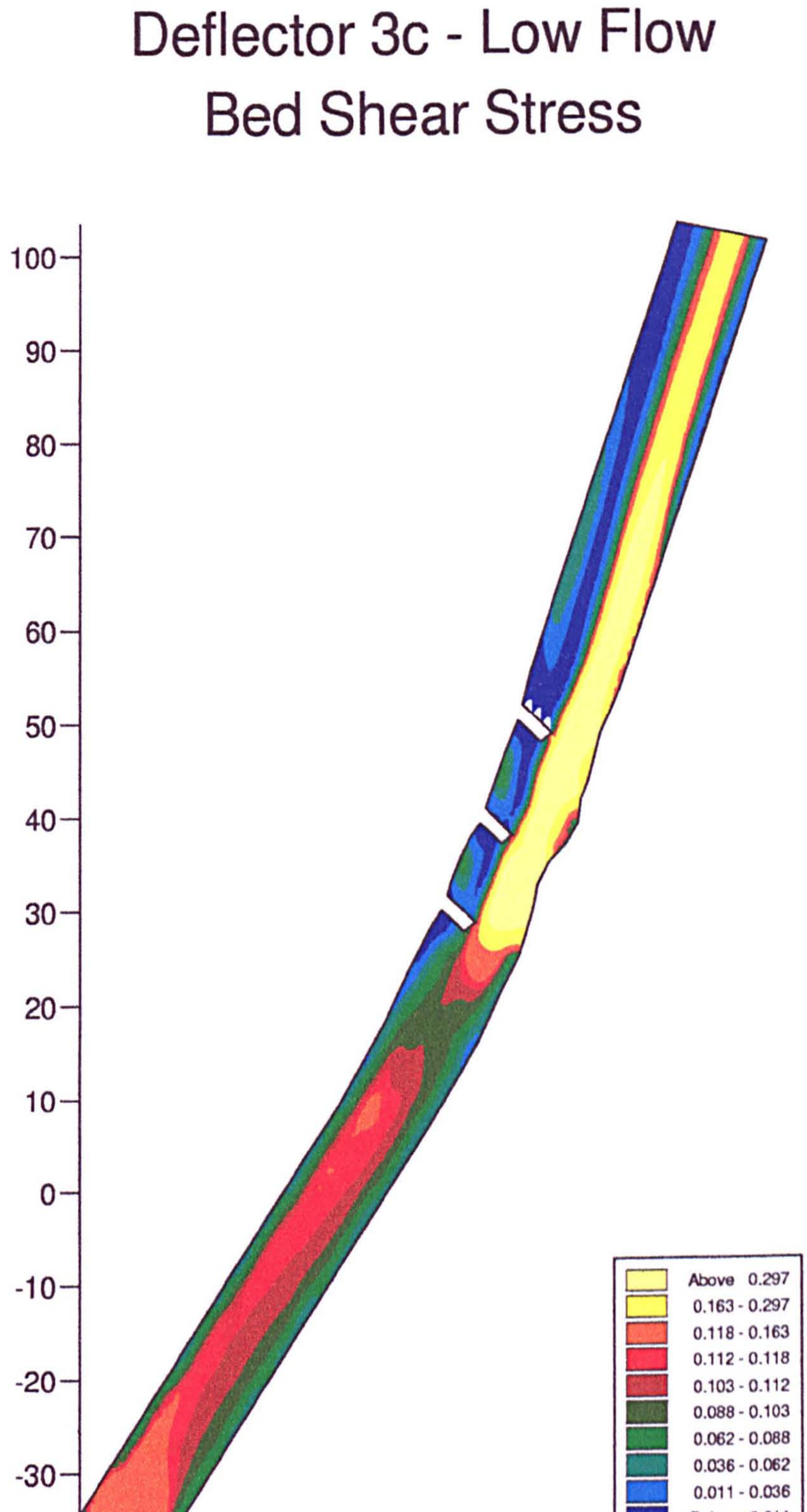

Deflector 3c - Low Flow
Bed Shear Stress
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.126
0.115 - 0.126
0.109 - 0.115
0.105 - 0.109
0.098 - 0.105
0.090 - 0.098
0.078 - 0.090
0.064 - 0.078
0.043 - 0.064
Below 0.043

Figure 6.7:



Deflector 3c - Low Flow
Bed Shear Stress
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-10
0
10
20
30
40
Above 0.297
0.163 - 0.297
0.118 - 0.163
0.112 - 0.118
0.103 - 0.112
0.088 - 0.103
0.062 - 0.088
0.036 - 0.062
0.011 - 0.036
Below 0.011

Figure 6.8:

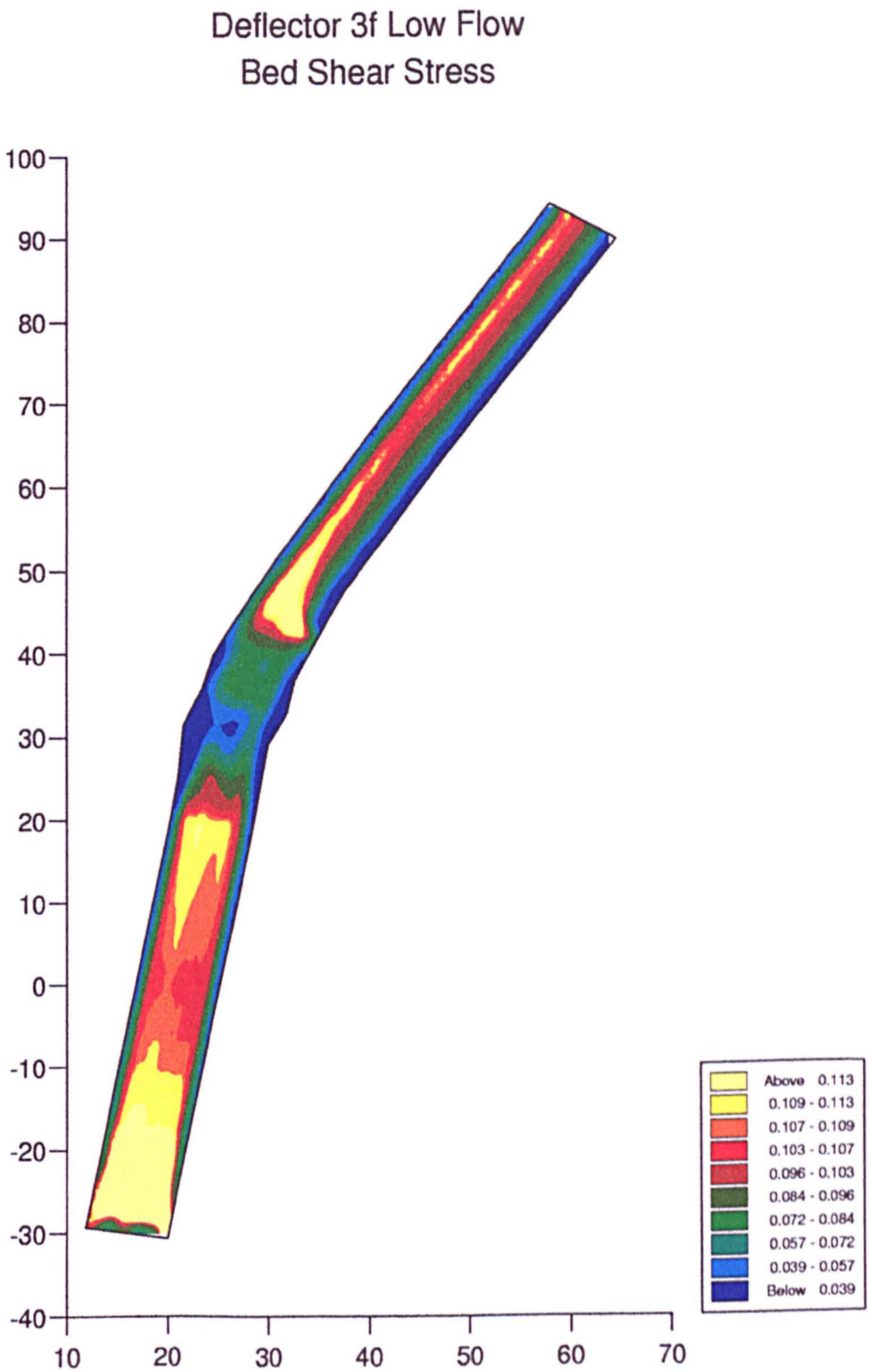

Deflector 3f Low Flow
Bed Shear Stress
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.113
0.109 - 0.113
0.107 - 0.109
0.103 - 0.107
0.096 - 0.103
0.084 - 0.096
0.072 - 0.084
0.057 - 0.072
0.039 - 0.057
Below 0.039

Figure 6.9:

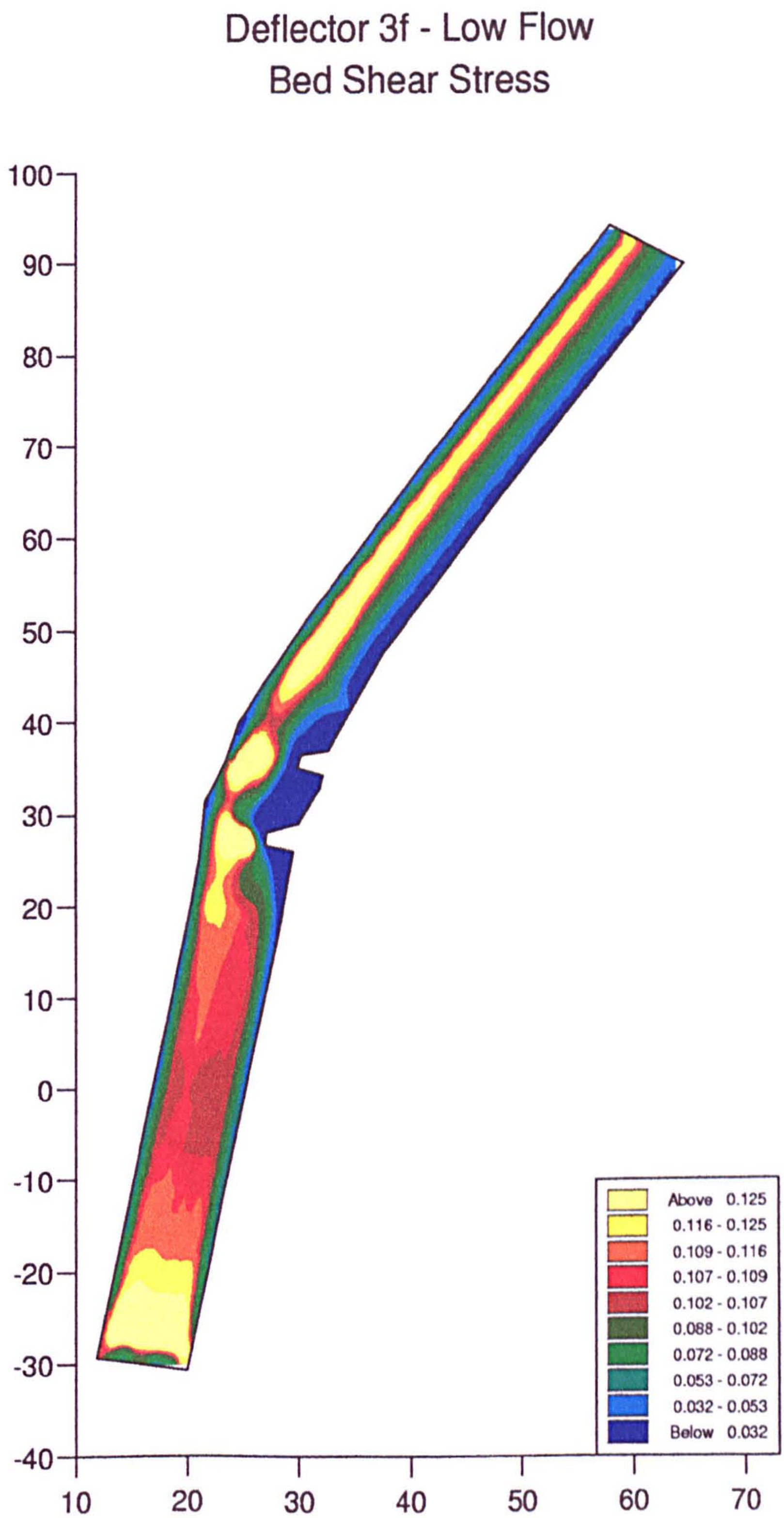
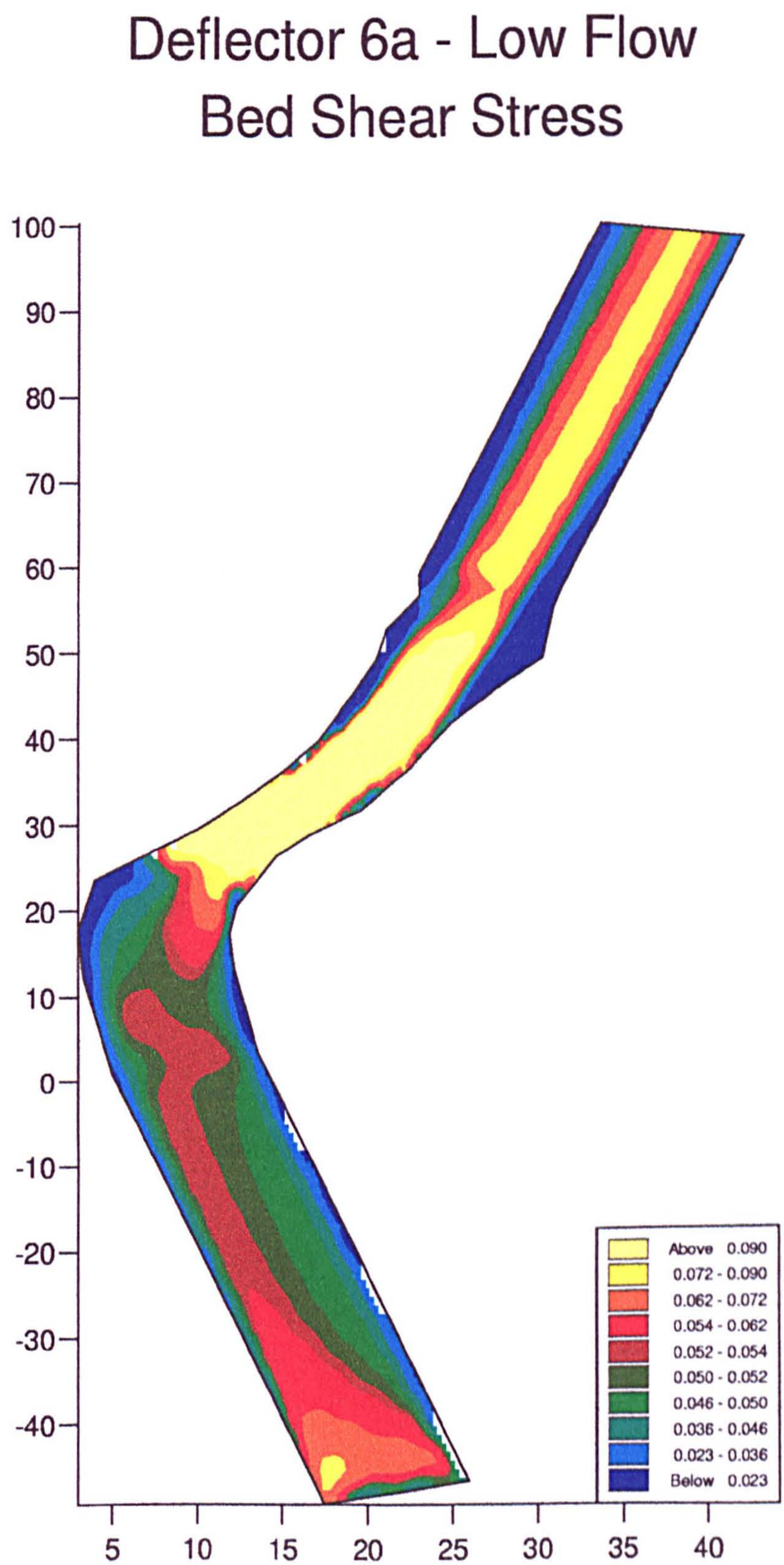
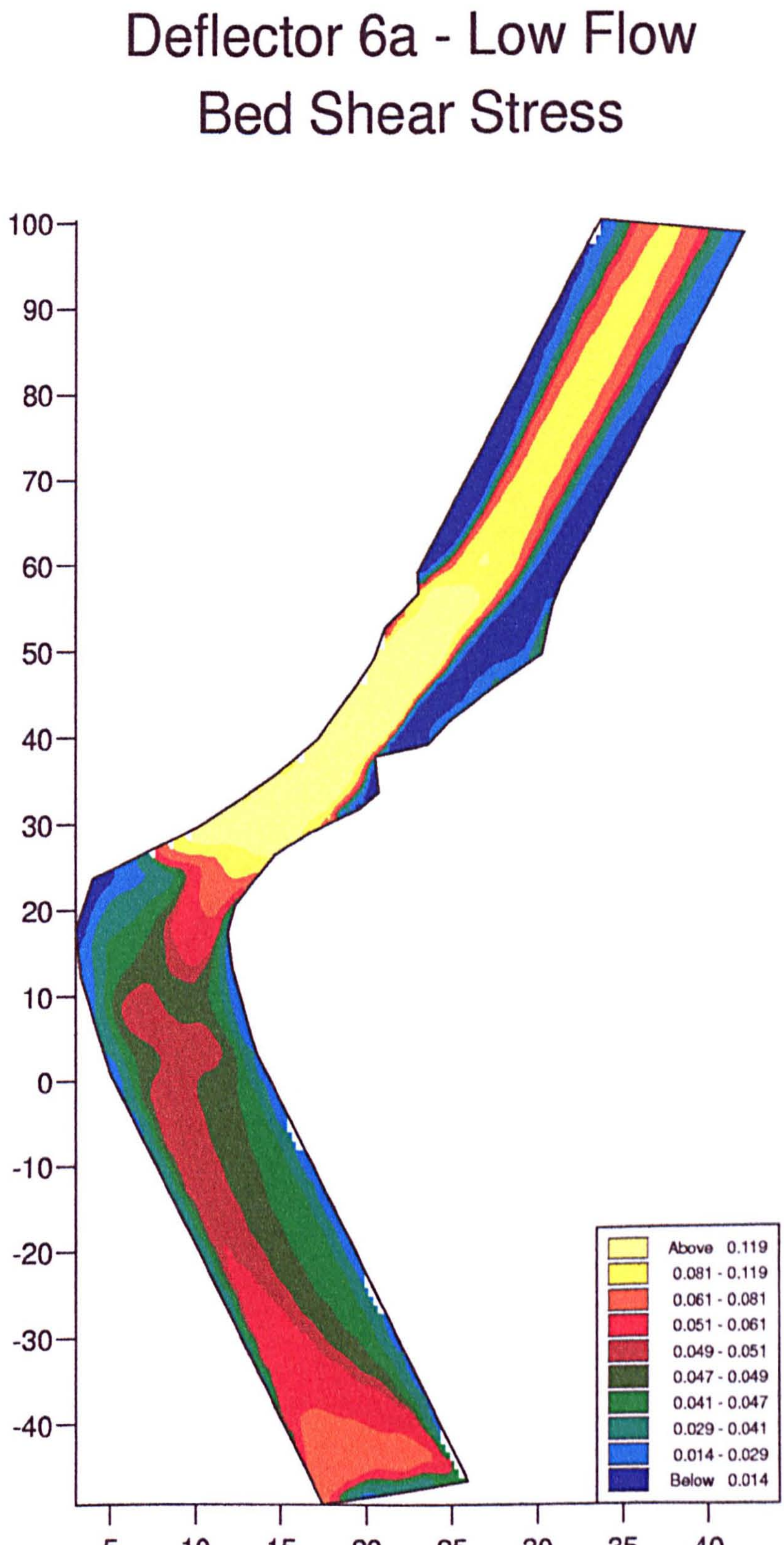

Deflector 3f - Low Flow
Bed Shear Stress
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70
Above 0.125
0.116 - 0.125
0.109 - 0.116
0.107 - 0.109
0.102 - 0.107
0.088 - 0.102
0.072 - 0.088
0.053 - 0.072
0.032 - 0.053
Below 0.032

Figure 6.10:



Deflector 6a - Low Flow
Bed Shear Stress
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
5
10
15
20
25
30
35
40
Above 0.090
0.072 - 0.090
0.062 - 0.072
0.054 - 0.062
0.052 - 0.054
0.050 - 0.052
0.046 - 0.050
0.036 - 0.046
0.023 - 0.036
Below 0.023

Figure 6.11:



Deflector 6a - Low Flow
Bed Shear Stress
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
5
10
15
20
25
30
35
40
Above 0.119
0.081 - 0.119
0.061 - 0.081
0.051 - 0.061
0.049 - 0.051
0.047 - 0.049
0.041 - 0.047
0.029 - 0.041
0.014 - 0.029
Below 0.014

Figure 6.12:

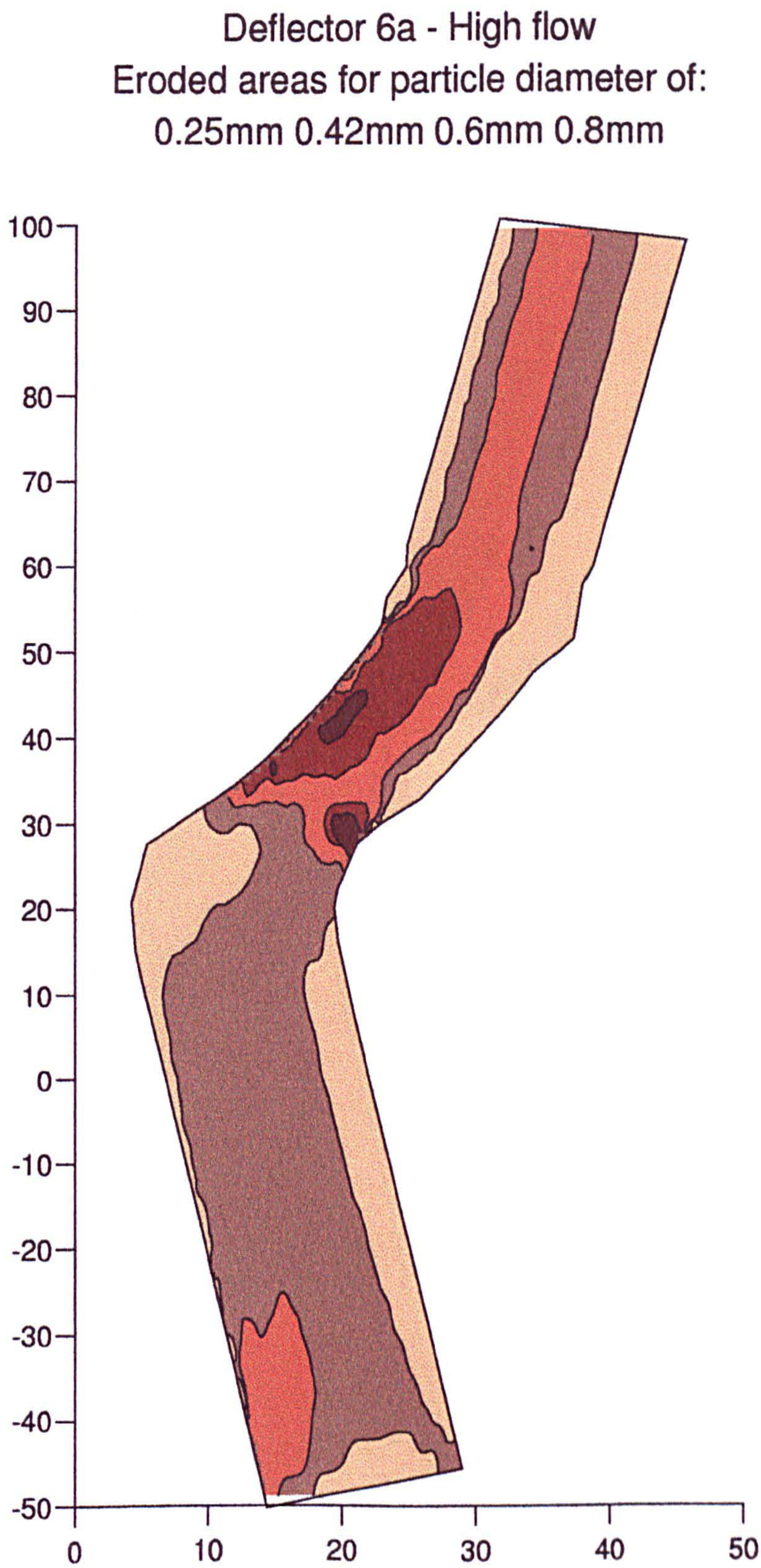

Deflector 6a - High flow
Eroded areas for particle diameter of:
0.25mm 0.42mm 0.6mm 0.8mm
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
0
10
20
30
40
50

Figure 6.13:

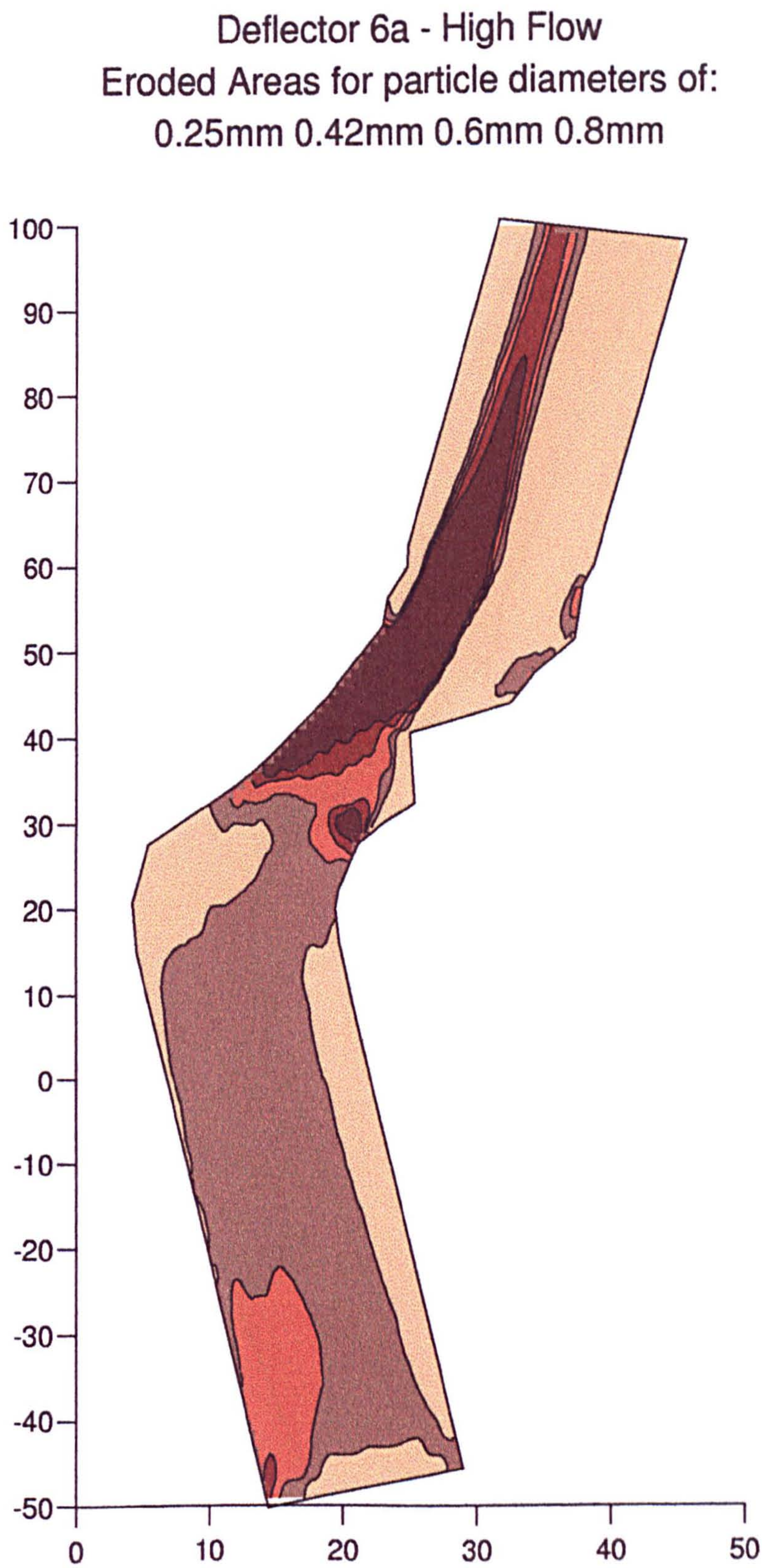

Deflector 6a - High Flow
Eroded Areas for particle diameters of:
0.25mm 0.42mm 0.6mm 0.8mm
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
0
10
20
30
40
50

Figure 6.14:

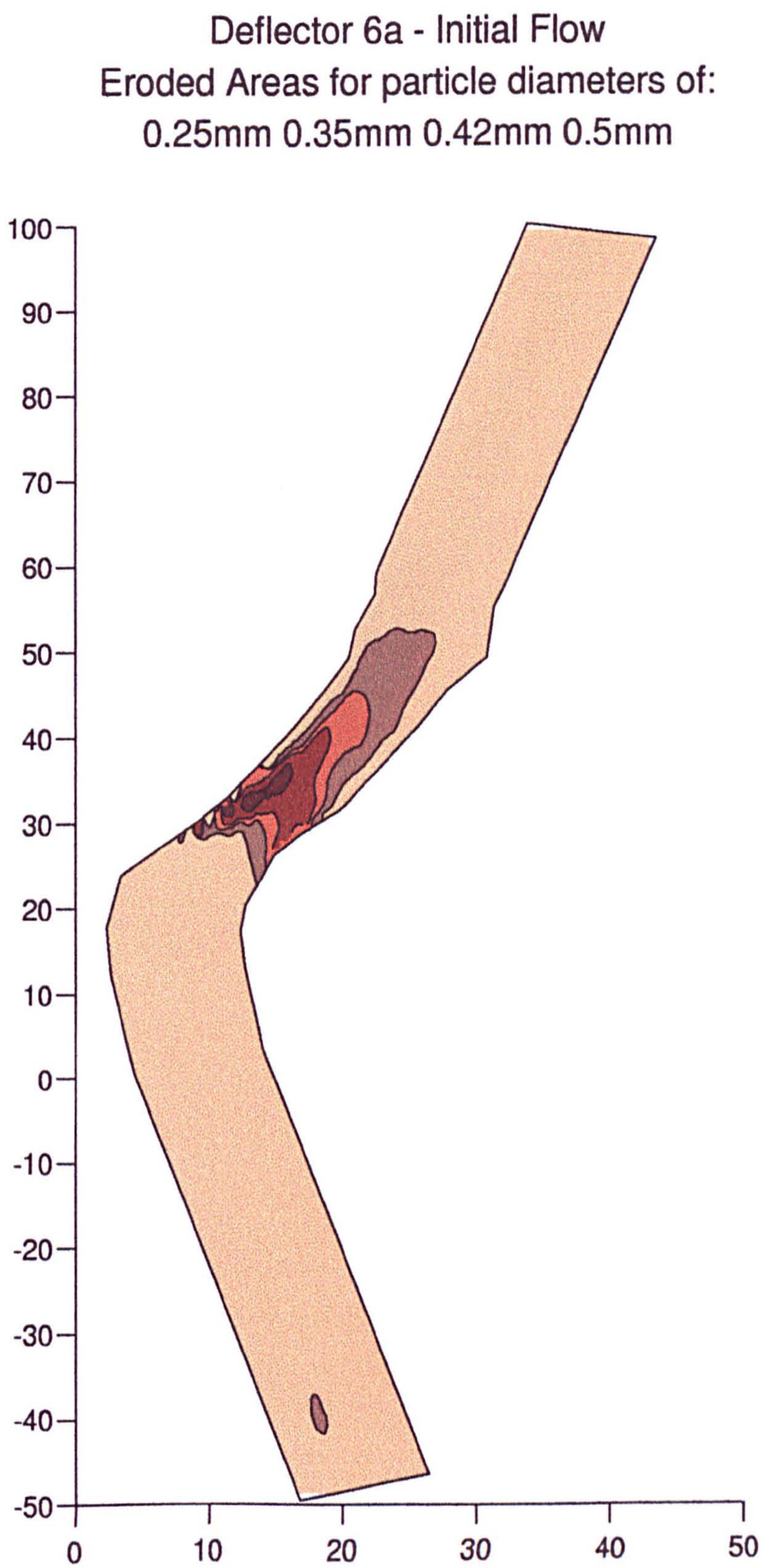

Deflector 6a - Initial Flow
Eroded Areas for particle diameters of:
0.25mm 0.35mm 0.42mm 0.5mm
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
0
10
20
30
40
50

Figure 6.15:

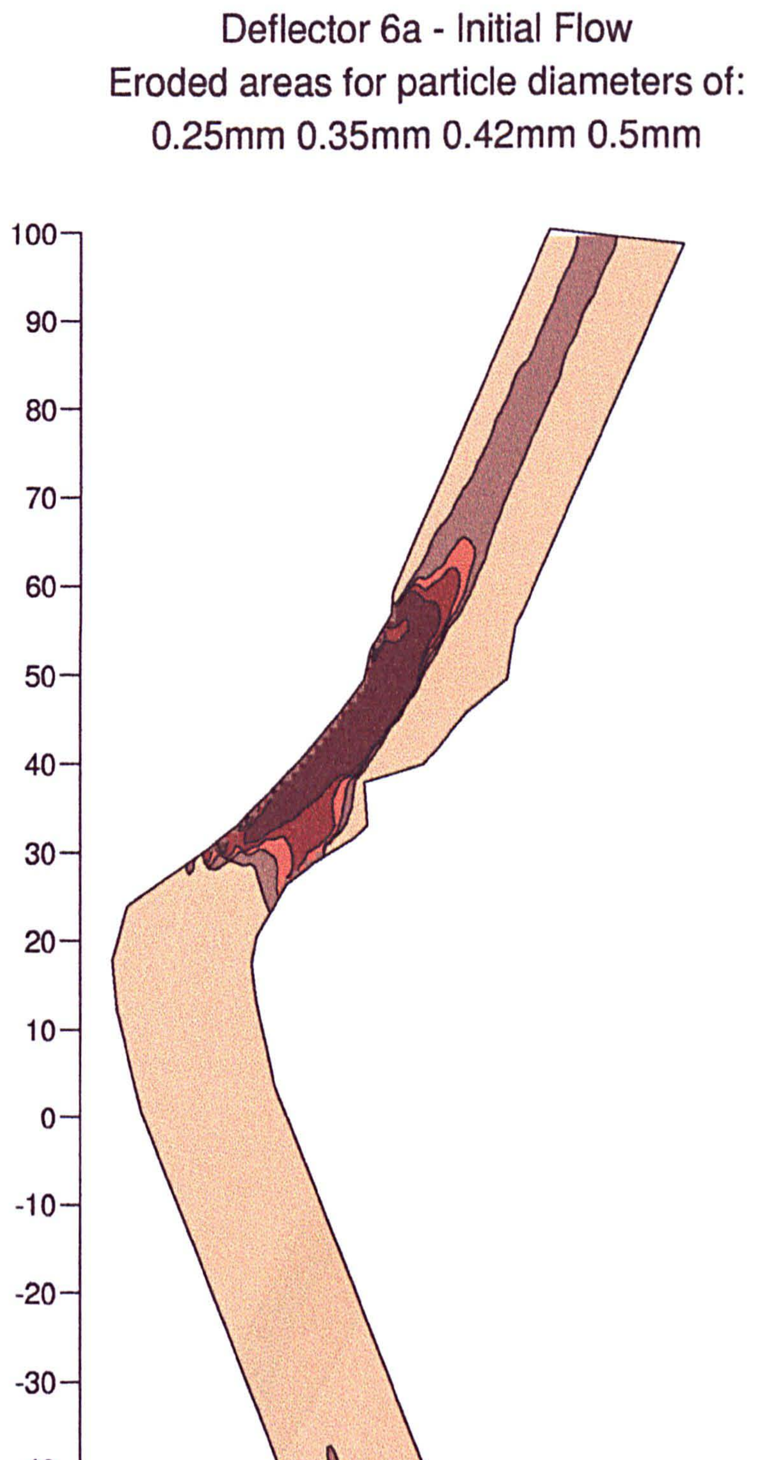
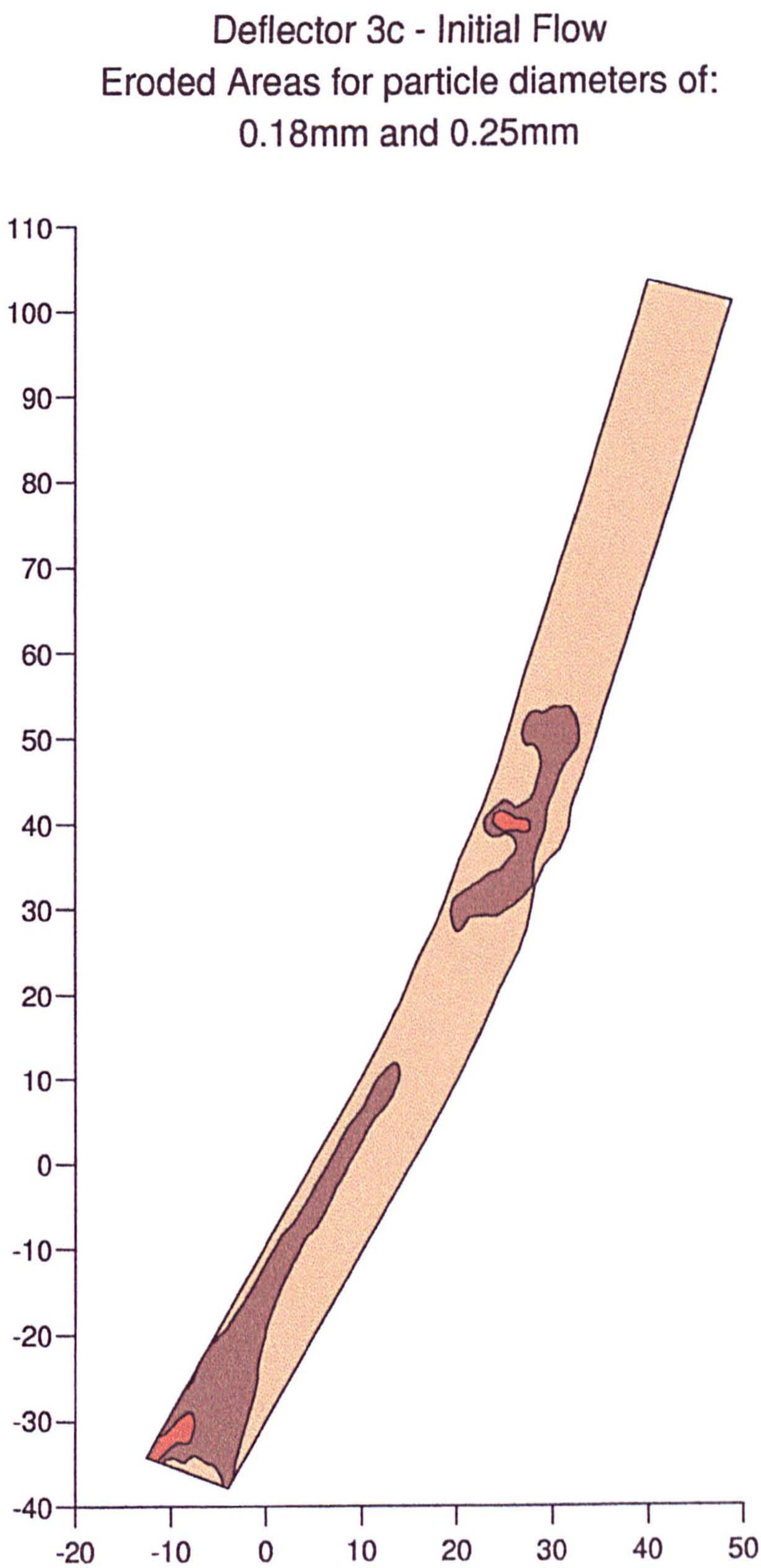

Deflector 6a - Initial Flow
Eroded areas for particle diameters of:
0.25mm 0.35mm 0.42mm 0.5mm
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
0
10
20
30
40
50

Figure 6.16:



Deflector 3c - Initial Flow
Eroded Areas for particle diameters of:
0.18mm and 0.25mm
110
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-20
-10
0
10
20
30
40
50

Figure 6.17:

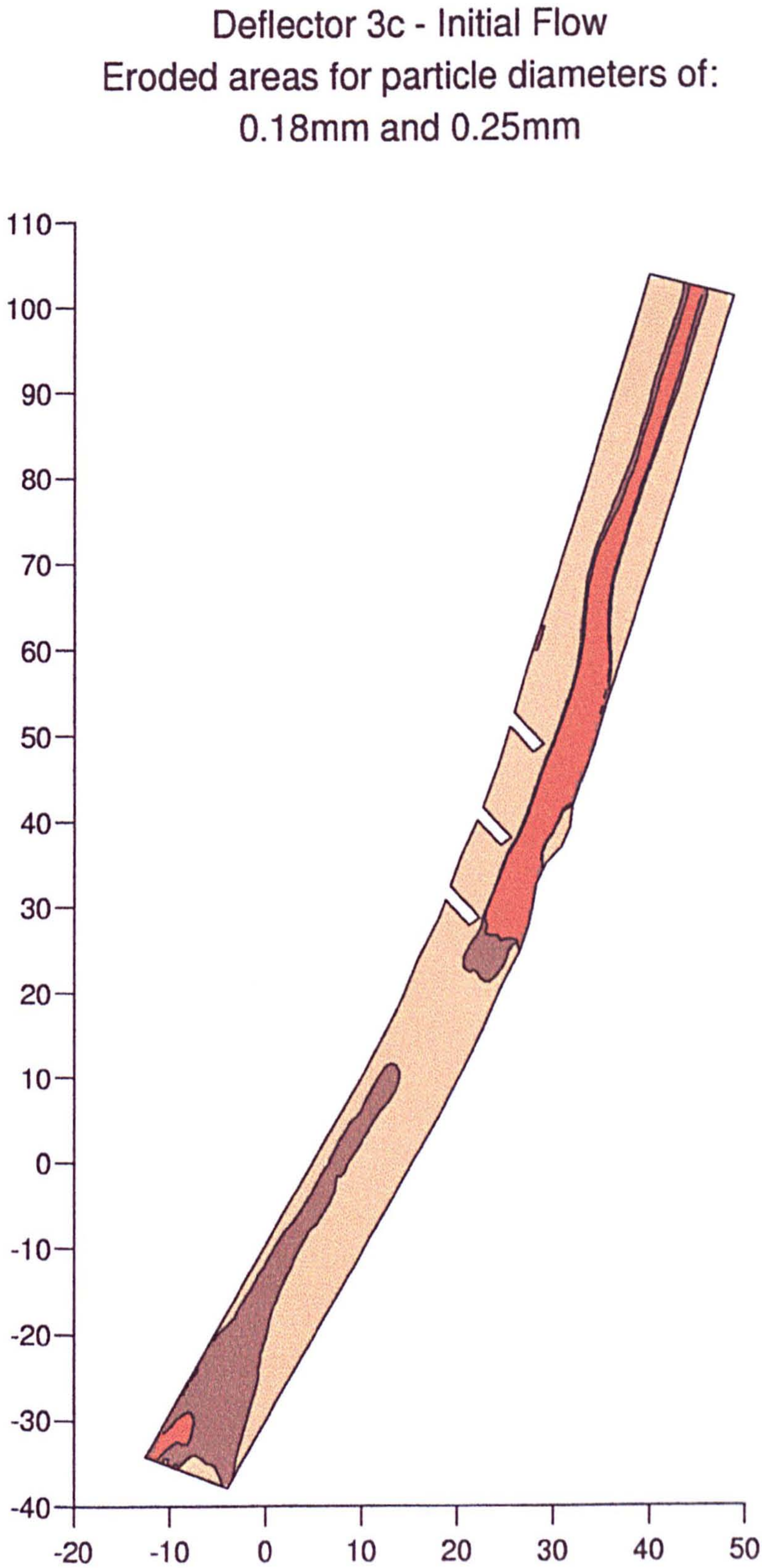

Deflector 3c - Initial Flow
Eroded areas for particle diameters of:
0.18mm and 0.25mm
110
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-20
-10
0
10
20
30
40
50

Figure 6.18:

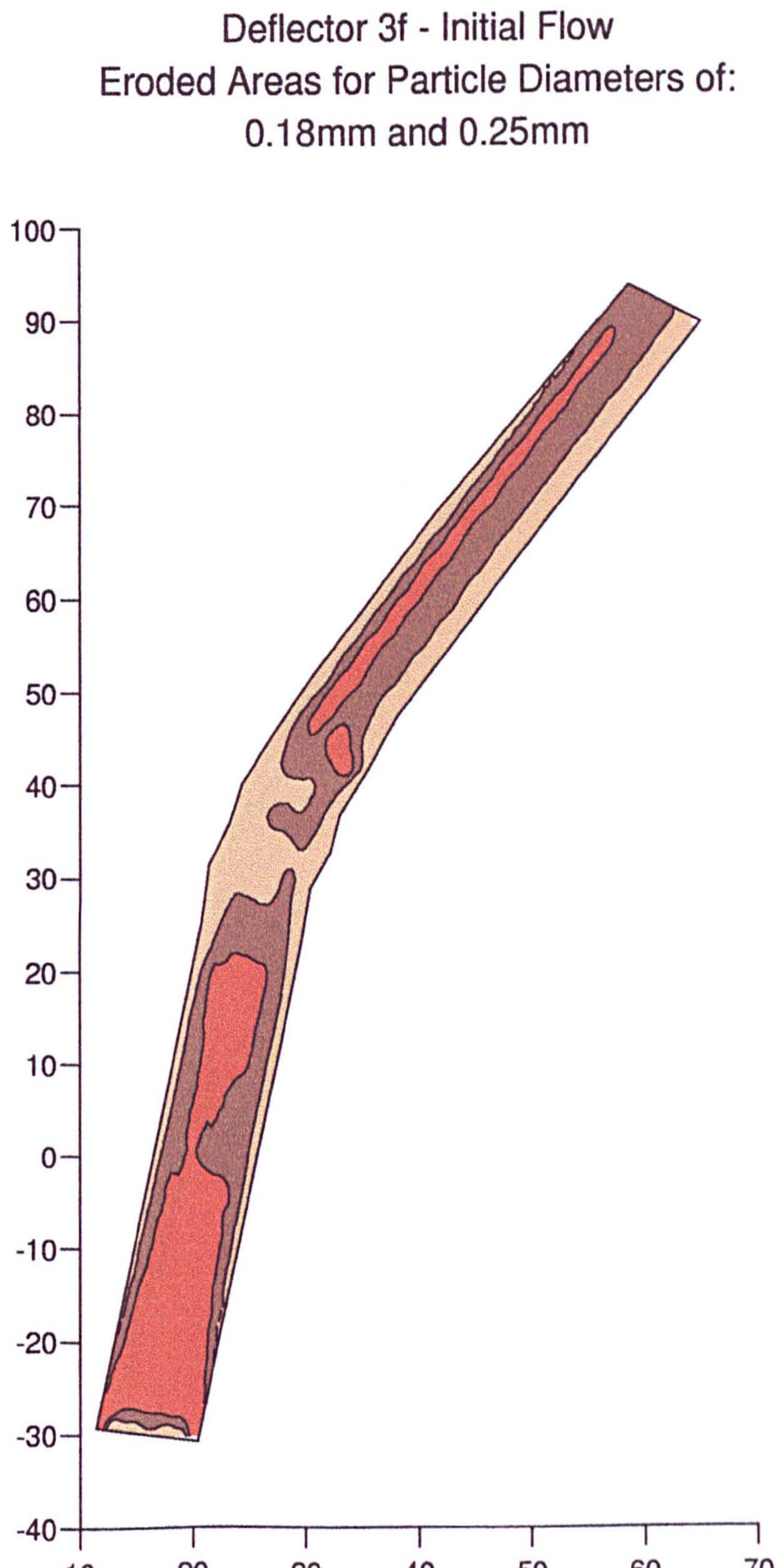
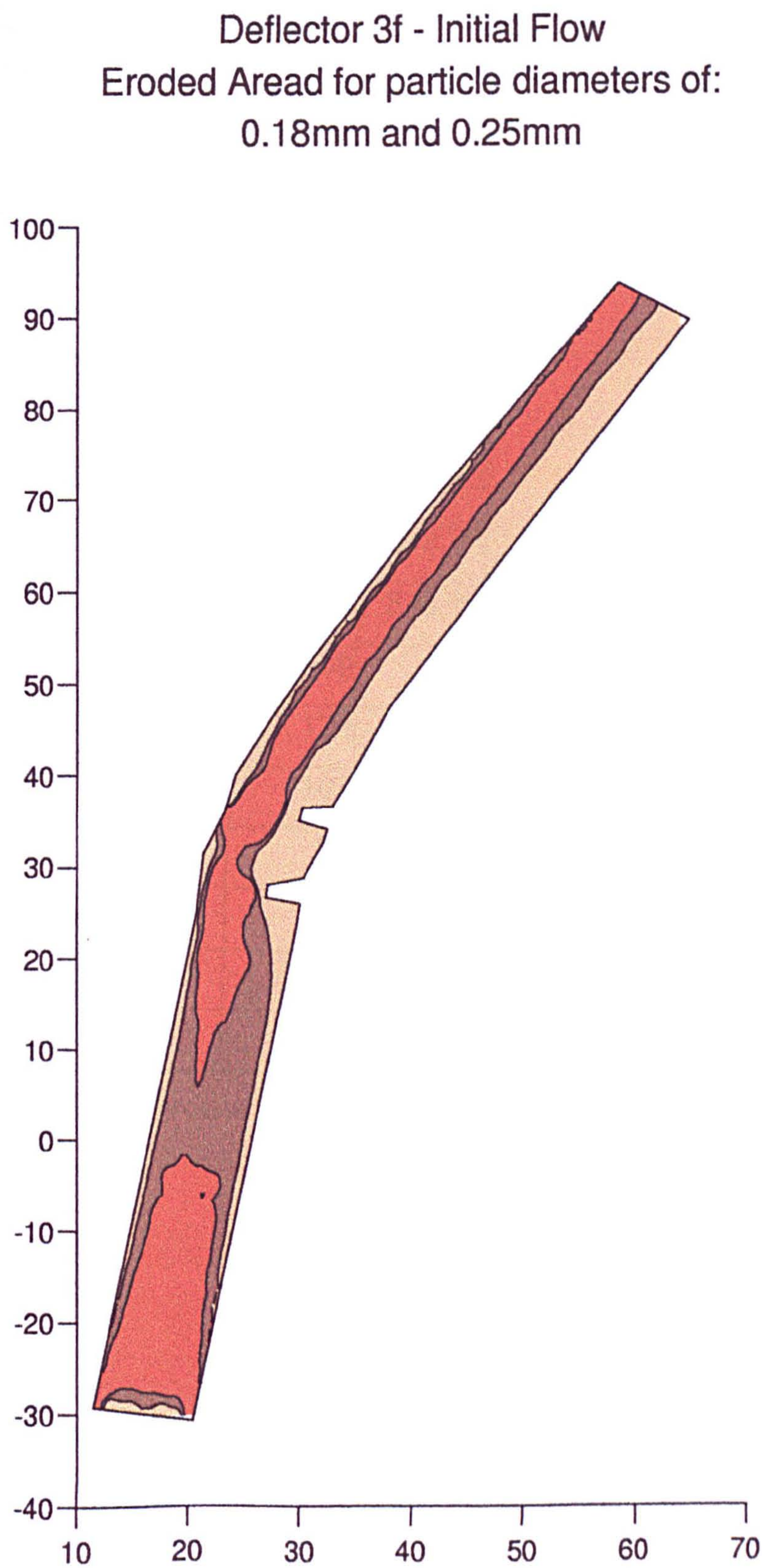

Deflector 3f - Initial Flow
Eroded Areas for Particle Diameters of:
0.18mm and 0.25mm
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70

Figure 6.19:



Deflector 3f - Initial Flow
Eroded Aread for particle diameters of:
0.18mm and 0.25mm
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
10
20
30
40
50
60
70

Figure 6.20:

Sediment diameter (mm)	6apre - Eroded area (m^2)	6apost - Eroded area (m^2)	% change
0.25	1524	1227	-19.49
0.35	914	726	-20.57
0.42	556	487	-12.41
0.5	274	374	36.50
0.6	142	297	109.15
0.7	64	245	282.81
0.8	21	193	819.05
0.9		159	
1		124	
1.1		103	
1.2		81	
1.3		68	
1.4		54	
1.5		36	

Table 6.1. Areas of erosion predicted by the program SHEAR at high flow for deflector 6a

Sediment diameter (mm)	6apre - Eroded area (m^2)	6apost - Eroded area (m^2)	% change
0.18			
0.25	168	310	84.52
0.35	84	168	100.00
0.42	45	135	200.00
0.5	10	95	850.00
0.6		63	
0.7		49	
0.8		39	
0.9		29	

Table 6.2. Areas of erosion predicted by the program SHEAR at the initial flow for deflector 6a

Sediment diameter (mm)	3cpre - Eroded area (m^2)	3cpost - Eroded area (m^2)	% change	3fpre - Eroded area (m^2)	3fpost - Eroded area (m^2)	% change
0.18	298	455	52.68	798.7	784.5	-1.7
0.25	23	219	852.17	351	466	68.42

Table 6.3. Areas of erosion predicted by the program SHEAR at the initial flow for deflectors 3c and 3f

Sediment diameter (mm)	6apre - Eroded area (m^2)	6apost - Eroded area (m^2)	% change
0.25	1	33	3200.00

Table 6.4. Areas of erosion predicted by the program SHEAR at low flow for deflector 6a

Sediment diameter (mm)	3cpre - Eroded area (m^2)	3cpost - Eroded area (m^2)	% change	3fpre - Eroded area (m^2)	3fpost - Eroded area (m^2)	% change
0.25	0.25	180	71900.00	0	0.64	

Table 6.5. Areas of erosion predicted by the program SHEAR at low flow for deflectors 3c and 3f

6.8.3 Site Measurements of bed elevation change

Plot 6.21 shows the recorded bed elevation changes for six cross sections at deflector site 6a for the period January 1996 to May 1998. Erosion is shown hatched in red and deposition in blue. The location of the six cross sections, in relation to the deflector, is shown in figure 6.22. The plot was provided by the School of Geography.

Plots for sites 3c and 3f have not been produced at the time of writing of this thesis. However, as in the case of deflector 6a, scour pools have been observed adjacent to the deflector tips and extending downstream in each case. In addition, deposition has been observed in the lee of each deflector.

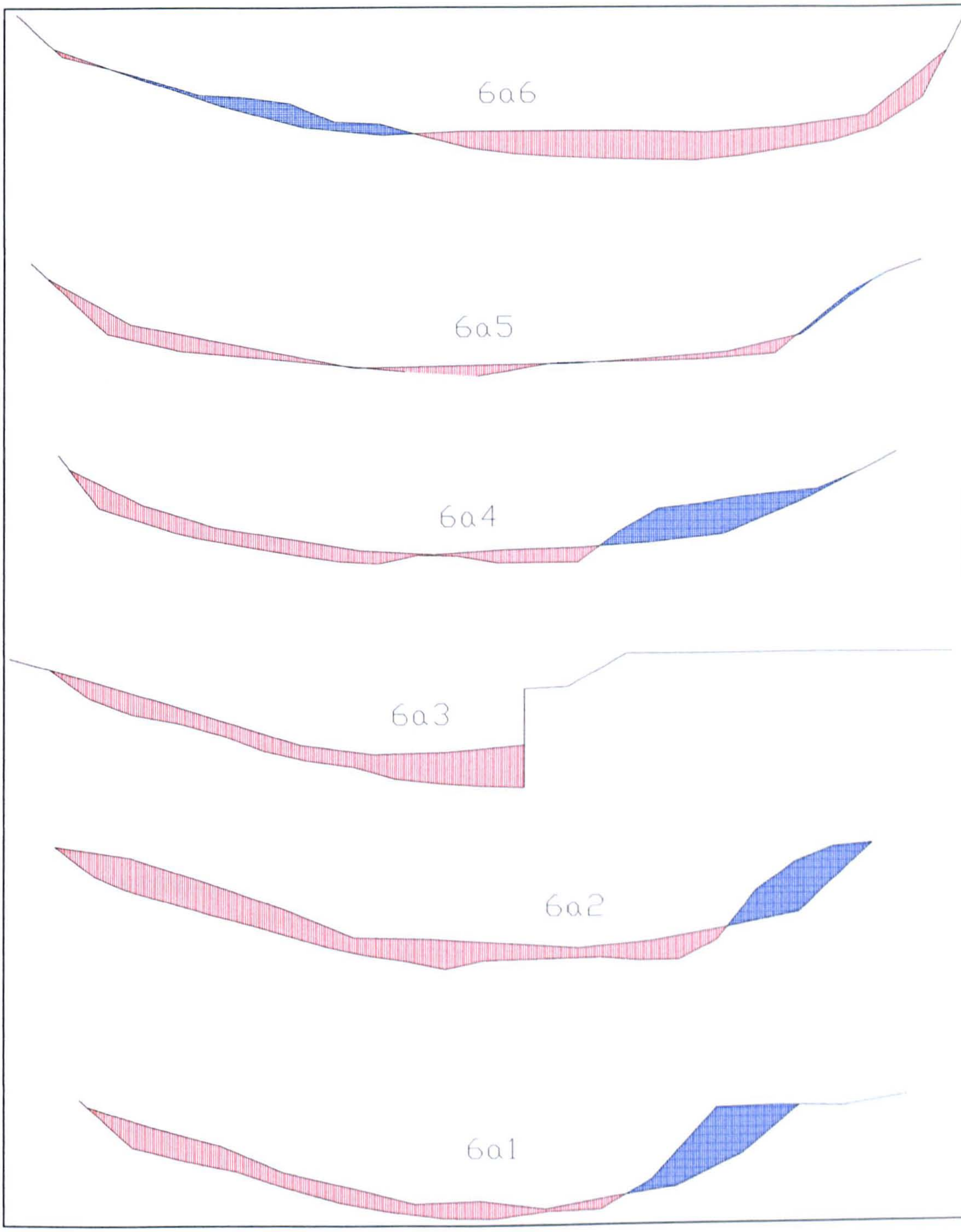


Figure 6.21. Bed elevation change at six transects for deflector site 6a as recorded on site between January 1996 and May 1998

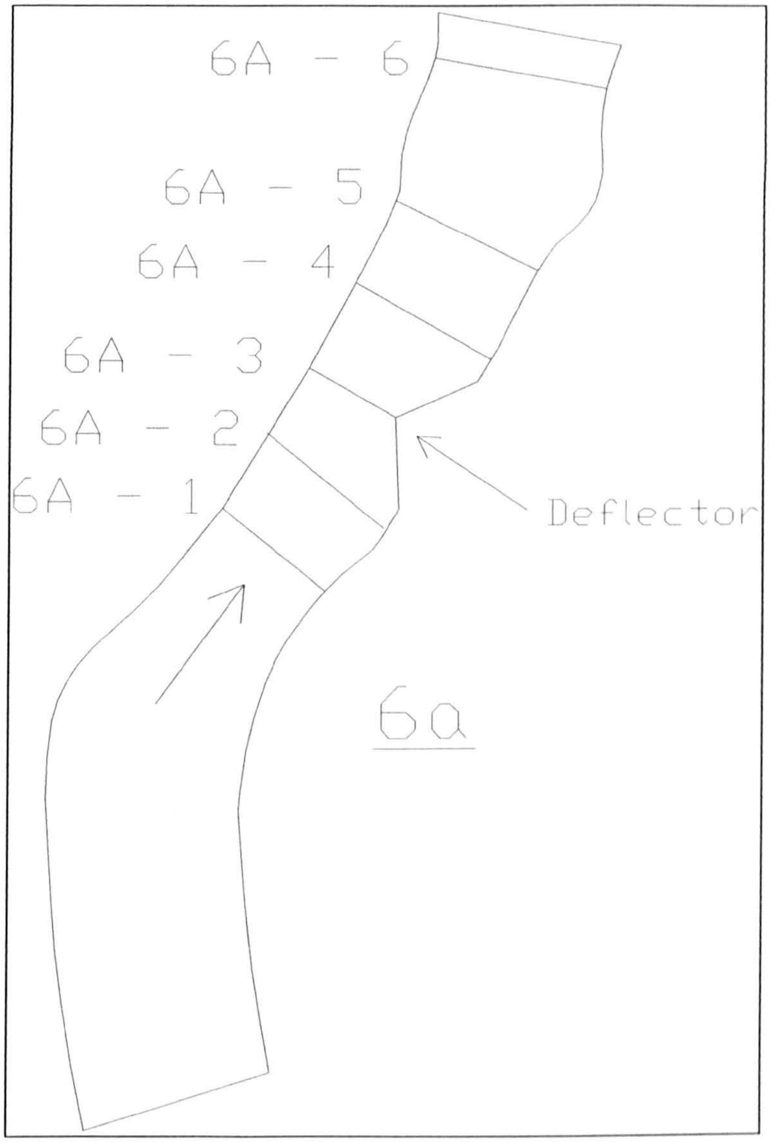


Figure 6.22. Location of the six transects at deflector 6a for which bed elevation change was measured.

6.9. Conclusions

6.9.1 One dimensional

The one dimensional results from the ISIS Sediment module show, in all cases, spikes of erosion forming at the locations of the deflectors. Thus, the method is successful in predicting that eroded pools will develop adjacent to the deflectors. The results produced have limitations in that they are only based upon a single discharge or a hydrograph for a single rainstorm. As a result the magnitude of erosion predicted to occur at the deflectors can not be taken as an accurate representation of that which is likely to occur on site. In order to accurately predict the magnitude of erosion at each deflector it would be necessary to carry out a simulation for the entire period since the deflectors had been installed, with the associated sediment and flow hydrographs. Even then this method would not predict the pattern of erosion accurately as it is severely limited by being one dimensional.

Erosion occurs at 3c and 3f at the comparatively low discharges that were present when the deflectors were surveyed (figures 6.5 and 6.6). At higher discharges more erosion could reasonably be expected. Erosion is also predicted to occur at each of the five deflector sites in section six, with a much higher discharge and for a much greater duration storm (figure 6.4).

A comparison of the results produced show that there are wide discrepancies in the amount of erosion predicted by the Engelund Hansen and the Ackers and White equations. Looking at the pre deflector situation for deflectors 3c, 3f and section 6 the Engelund Hansen formulae predict 1609, 1591 and 986,294 kg of erosion to take place through the modelled sections respectively. The Ackers and White formulae predict 215, 238 and 209,624 kg of material to be eroded for the same reaches. This indicates the wide discrepancy in values that can be attained by using different sediment formulae.

The magnitude of the erosion peaks are also fairly dissimilar for the two methods, although not to the apparent extent of the predicted weights of eroded material. The Engelund Hansen approach predicts erosion peaks of 0.25m, 0.48m and 0.16m for the three wing deflectors that together make up deflector 3c (figure 6.5). The Ackers and White formulae predict peaks of 0.23m, 0.18m and 0.22m respectively.

At deflector 3f the Engelund Hansen approach predicts peaks of 0.32m and 0.15m (figure 6.6). The Ackers and White formulae predict peaks of 0.2m and 0.12m. For section six, the five erosion peaks at the deflectors are 0.6m, 0.54m, 0.1m, 0.12m and 0.01m with the Engelund Hansen formulae. With the Ackers and White formulae the corresponding peaks are 0.46m, 0.44m, 0.1m, 0.2m and 0.05m (figure 6.4). This highlights the fact that the Ackers and White formula predicts less erosion than Engelund Hansen in the cases modelled here.

Considering the weight of material that is transported out of each of the modelled sections, the results show that the installation of the deflectors causes this amount to rise only by a tiny fraction. In all cases the increase in weight of material that is transported out of the modelled reach rises by less than 1.3%. In most cases the rise is less than 0.1% and in the lowest case it is 0.01%. This suggests that the extra material that is eroded by the deflectors is deposited immediately downstream. This conclusion is backed by considering the changing profile of the predicted bed elevation change.

After the deflectors are installed, increased heights of deposition can be observed in the lee of the deflectors. For example, at deflector 3c the Engelund Hansen formulae predicts that the bed height will rise by 0.06m immediately downstream of the second wing deflector. Prior to the deflectors, the rise in bed height at this location is only 0.03m. With the Ackers and White formula 0.02m of deposition is predicted following the deflector installation as compared to 0.01m beforehand. The pre deflector long section plots for 3f and 3c have not been included because they appear as virtually horizontal lines, implying very little erosion or deposition over the short period simulated

6.9.2 Three Dimensional

For deflector 3c, the maximum bed shear stress increases from 0.18 N/m^2 to 0.314 N/m^2 at the initial flow level and from 0.126 N/m^2 to 0.297 N/m^2 at the low flow level. The low flow result is shown in figures 6.7 and 6.8. This represents rises of 74% and 136% respectively.

For deflector 3f, the maximum bed shear stress increases from 0.239 N/m^2 to 0.28 N/m^2 at the initial flow level and from 0.113 N/m^2 to 0.125 N/m^2 at the low flow level. The low flow result is shown in figures 6.9 and 6.10. This represents rises of 17% and 11% respectively.

The results for deflector 6a show that the installation of the deflectors increases the bed shear stress from a maximum of 0.48 N/m^2 to 0.672 N/m^2 for the high flow, from 0.242 N/m^2 to 0.361 N/m^2 at the initial flow level, and from 0.09 N/m^2 to 0.119 N/m^2 at the low flow level. The low flow result is shown in figures 6.11 and 6.12. This indicates increases in bed shear stress of 40%, 49% and 32% respectively.

It should be noted that it is the actual values of shear stress that determine whether erosion will occur at any location. In these terms, the highest shear stresses at low flow (which is the only comparable discharge here since the initial flow level represents a different discharge at each site) occur at site 3c. Thus, deflector site 3c is expected to be the most successful in generating erosion.

The percentage increases give an indication of the effectiveness of the deflectors in increasing the bed shear stress, and therefore generating scour pools. This must be taken in context with the corresponding results plot which represents the pattern of bed shear stress, since the percentage increases relate only to one value: the maximum. Again, the largest increases occur at 3c.

At the high discharge ($8m^3/s$) modelled on deflector 6a, it can be seen that the effect of installing the deflector was sufficient to cause bed erosion to take place opposite the deflector with a particle diameter as high as 1.5mm (table 6.1). Although this is a very high discharge for the Idle, and will occur very rarely, it does suggest that large amounts of sediment will be mobilised at this time. Particle diameters up to 0.9mm would be eroded adjacent to the deflector at 6a, even at the comparatively low 'initial flow' discharge of 1.7m^3/s (table 6.2).

Considering the eroded areas of bed, shown in table 6.1, it is noticeable that at the high discharge modelled on deflector 6a, the effect of the deflector is to reduce the amount of erosion that takes place for the smaller grain diameters. This is due to the fact that at high discharges the deflector constricts the flow and reduces velocities upstream where they would otherwise be large enough to cause erosion. In addition, the relatively lower velocities produced in the recirculation are insufficient to cause erosion. In contrast, in the region opposite the deflector the velocities are increased and this produces a bed shear stress which is large enough to induce erosion for the larger particle diameters where little or no erosion would otherwise take place.

At lower discharges the effect of the deflectors is particularly pronounced. Before the deflectors were installed the flow was very uniform and there was insufficient bed shear stress, throughout the areas under consideration, to erode the bottom. The effect of the deflectors is to produce a localised area of relatively fast flowing water, which generates a small zone of high bed shear stress. At low flows, the relatively small area of erosion that is predicted is great in proportion to the amount that took place beforehand when there were no areas of fast moving water. The effect is highlighted by plots 6.16, 6.18 and 6.20 which indicate significant increases in eroded areas of bed. This is manifest in the large percentage rises shown in tables 6.2 to 6.5. The results show that the area of erosion can be several thousand times higher after the deflectors are installed. This is important as it suggests that the shear stresses produced opposite the deflectors are probably sufficient to continue to remove any further sediment that is deposited there (and thus maintain a clean bed of coarse material as a spawning area) even at relatively low flows.

Deposition of material is inferred from the result plots by low values of bed shear stress. This is indicated by the dark blue shading on the graphs. From the results, it appears that deflectors 6a and 3c are probably the most successful in causing a zone of low shear stress in the lee of the deflector. These zones can be seen extending for many metres downstream in the flow in plots 6.8 and 6.12. Deflector 3f produces a zone of low shear stress between the two deflectors, but relatively less downstream as shown in plot 6.10. Thus it appears to be less likely to induce sediment to be deposited in the lee.

Another factor that will contribute to whether sediment is deposited in the lee of the deflector is the flow pattern in that area. If a slow recirulation is predicted then transported material in the flow behind the deflector will be entrained within that zone for a longer period of time. As a result, particles will have more opportunity to be deposited. If no recirculation occurs, then transported material that passes through the area behind the deflector will only be resident in that area for a small period of time. As a result, there is much less opportunity for it to be deposited in that zone. In the case of the Idle, flow recirulations are predicted for all of the deflector sites as shown in the results plots in chapter 4. This indicates a high likelihood that sediment will be deposited there.

In reality, spatial modelling of the deposition of sediment can only be performed accurately by assessing the transport capacity of the flow and sediment continuity as ISIS Sediment does. An analysis of this type in two or three dimensions would be highly complex and is beyond the scope of this thesis. The three dimensional approach used here is much simpler, and focuses on the effect of discrete values of discharge on the pattern of bed shear stress.

When considering the results given in this chapter, it should be borne in mind that the flow is developing over the first section of the reach, on the approach to the deflector. The reason for this is discussed in chapter 4. Thus, plotted results for bed shear stresses and eroded areas are incorrect in this area. The results given should only be considered from the region immediately above the deflector and downstream of this point. Plots are given for the entire region that was modelled for completeness, and so that all of the results can be seen. In addition, it is difficult to plot only part of the region for which input data is included, with the data visualisation package Gsharp that was used here.

6.9.3 Validation of sediment modelling results

Unfortunately, it is not possible to validate the results from the one and three dimensional modelling in a quantitative way, for reasons that have already been discussed. However, it is possible to make some comments on a qualitative basis regarding the success of the two methods in predicting the changes recorded on site for deflector 6a as shown in figure 6.21. Site observations suggest that the overall pattern of erosion and deposition is similar at deflectors 3c and 3f.

Results from the one dimensional modelling show peaks of erosion at the deflector tips (figure 6.4), and this effect is readily confirmed by the site measurements, which show that erosion has taken place adjacent to the tip of deflector 6a (figure 6.21). The results from the SHEAR program, that make a more detailed estimation of the likely areas of bed erosion, are more difficult to assess. Certainly the major feature of the site measurements is reproduced: namely the erosion towards the left hand side of the channel which extends both upstream and downstream of the deflector. This can be seen at sections 6a1 to 6a5 in figure 6.21. This corresponds well with the increased area in which erosion is predicted by SHEAR (for each of the particle diameters modelled), as shown in figures 6.16.

The deposition that has been seen to occur behind the deflector (figure 6.21 section 6a4 and 6a5) can also be successfully implied from figure 6.12, which shows a zone of reduced bed shear stress in the deflector's lee. The deposition that has occurred upstream of the deflector, adjacent to the right hand bank, (figure 6.21 section 6a1 and 6a2) is somewhat less well predicted by SHEAR, although there is a small zone of low shear stress in the area mentioned on figure 6.12. Overall, the comparison between the site and simulated bed changes appears to be very encouraging.

6.9.4 Overall

One of the main features identified in the geomorphological survey of the Idle was the presence of large amounts of sand sediments on the bed of the channel which were blown across from adjoining agricultural land, The existing transport capacity of the river was insufficient to erode this sediment downstream. In addition, the stream power of the channel was found to be very low, which would explain why the channel had not significantly adjusted following channelization. The design of the deflectors was intended to develop small areas with increased transport capacity where the bed would be eroded to form scour pools with a cleaner gravel bed. At the same time, it was important not to create excessive instability. This was unlikely due to the very low stream power of the channel.

A comparison of the results produced here with some of the initial findings from the site surveys of the Idle, following the deflector's installation, shows good agreement, albeit in a qualitative sense. The major features found in the site measurements of bed change are the erosion of the sand substrate to form a scour pool abreast of each of the deflectors and extending for several metres downstream. In addition, deposition in the lee of the deflector has created a bank of sediment in each case. A more detailed comparison of the modelling results with site observations will have to await the publication of the full investigation into the deflector's effect. This report is due from the School of Geography this autumn or winter.

When considering the observed effects of the installation of the deflectors the prevailing discharges pre and post rehabilitation should be borne in mind. These factors are rarely given much consideration, but are in fact of great importance. If deflectors are installed immediately after a 100 year flood event, for example, then the river cross section is very likely to have been eroded and widened by the flood. As a direct result, depositional processes are likely to ensue for a period to return the river to something approaching its original cross section. The effect of the deflectors may be masked by these processes.

The prevailing discharges after the deflectors have been installed are also of great importance. If a 100 year flood occurs soon after construction is complete, then large amounts of erosion are likely to take place which will again mask the effect of the deflectors.

In the case of the rehabilitation of the River Idle, the prevailing discharges have been lower than expected since the deflectors were installed. As a result, it is possible that the full effect of the deflectors has not yet been seen. These may not be realised until a significant flood peak is produced.

The detailed sediment modelling used two approaches. The one dimensional approach uses sediment transport formulae whilst the three dimensional approach relies on assessing bed shear stresses. Both approaches have advantages and limitations. In conjunction with a detailed geomorphological survey, the methods provide a viable means of predicting channel change. If the results from the geomorphological survey and the detailed modelling using the twin approach were available at the design stage it would have been possible to predict, with some degree of detail, the effect that the installation of the deflectors would have had. The morphological survey identified the main cause of the problem, and the fact that the river was unlikely to react markedly to rehabilitation. The detailed modelling accurately described the location and extent of the erosion and deposition.

6.10. Discussion

The results from both ISIS Sediment and the three dimensional SHEAR program are based on only one grain size which corresponds to a medium sand. The bed substrate is composed of around 75% of medium sand so this assumption is not a bad one. In reality, smaller percentages of other grain sizes are present and these will have an effect on sediment transport and the erosion of the bed. In particular, the presence of a variety of grain sizes helps to produce an armouring effect whereby the particles interlock to produce a greater resistance to motion. This effect is complex to simulate and is not included in either model.

It would have been interesting to carry out, as part of this thesis, some three dimensional modelling of the deflector sites with the post scheme surveyed cross sections. The same cross sections that were used to create the flow models were resurveyed several months after the installation of the deflectors. At this time the scour pool at the deflector tip, and a deposition area in the lee of the deflector, had begun to form. Modelling the deflectors with the altered bed formation would have given a good picture of the changing pattern of shear stress at the deflector sites. The results from the three dimensional simulations using the pre scheme bedforms were very successful in predicting the high shear stress, and subsequent erosion, adjacent to the deflector tip and the low shear stress, and subsequent deposition, behind the deflector. It would have been useful to see how these shear stresses had changed once the scour pool and deposition area had formed. The predicted shear stresses would have been useful in predicting if the scour pool was fully formed, or if the bed shear stresses were still large enough to cause erosion. However, once the scour pool began to erode a gravel substrate was exposed underlying the sand. Accurate sampling of this substrate would have been required to determine its critical shear stress, and at what depth under the sand it exists. It would have been hard to include in the model where the gravel was exposed and where the bed was composed of sand, so that the relevant critical shear stress could be applied to determine if erosion took place. In addition, gravel has a greater roughness than sand, so for complete accuracy it would be necessary to include a different wall roughness in the flow model in the region where the gravel substrate was exposed. This is not easily achieved within CFX.

Chapter 7

Conclusions

7.1. Main Findings

This work used sophisticated hydraulic modelling techniques to examine the effect of a river rehabilitation scheme from the three key perspectives of flooding potential, aquatic habitats and the stability of the modified channel. The approach is novel in that one, two and three dimensional hydraulic models were used. The approach in industry utilises one dimensional models only. As a consequence, the results presented here give an important indication of the benefits that could be obtained with a more detailed approach. The modelling packages used in this thesis are: ISIS, HEC-RAS, SSIIM and CFX. FORTRAN programs were written by the author to process the results from the various packages to predict habitat change, the movement of sediments, and to validate the predicted velocities against site measurements.

The River Idle was used as the basis for this study. The Idle was channelized in 1978, and subsequently rehabilitated in 1996. The major part of the rehabilitation consisted of the construction of a number of flow deflectors. The purpose of installing these deflectors was to produce local variations in the channel width and flow velocity. The increased flow variability was intended to produce improvements in aquatic habitat, and also lead to the development of a more natural pool riffle sequence.

The major conclusion to draw from the results is that three dimensional models offer the possibility to predict changes in the aquatic habitat more accurately, and therefore more reliably, than the traditional one dimensional approach. Predictions from the one dimensional model underestimate habitat improvements due to the fact that only one momentum equation is solved. As a result, the local increases in velocity arising from the rehabilitation become averaged out over the whole of the channel width. With the three dimensional model, local velocity increases are not averaged out so the habitat improvement is accurately predicted.

The approach of using the predicted velocity profile from CFX to estimate bed shear stresses, detailed in chapter 6, also highlights the potential advantages of a three dimensional approach. Using this technique it is possible to map areas of the bed where erosion is predicted in great spatial detail. This gives a good indication of the effect of the rehabilitation on channel stability.

The application of two and three dimensional hydraulic models to river rehabilitation schemes is limited by the lack of a package designed specifically for the river modelling community. If such a package becomes available, river modellers should be encouraged to take advantage of the new technology so that the potential benefits described here can be used to design better river rehabilitation schemes.

Some of the main findings arising from this thesis are now discussed in more detail, together with recommendations for future research on this study and in the field in general

7.1.1 Flow modelling

Flow modelling, using one dimensional packages, to determine the effect of installing deflectors in the Idle on flood levels is carried out in chapter 4. Results show that the largest increase in flood stage is only 0.13m. With normal levels of maintenance, the largest increase in flood stage would be reduced to 0.05m. Thus, the installation of the deflectors does not cause a significant worsening of flood risk to surrounding land.

In the same chapter, the velocities predicted by SSIIM and CFX are calibrated against site measured velocities. Unfortunately, the validation process is hindered by the fact that site velocities measured with a two dimensional electromagnetic flow meter were only taken at one deflector site (3f). Even here a total of only fifty site velocities, at regular spatial intervals around the deflectors, were measured. At deflector site 3f, 85% of the CFX predicted velocities are found to be within 50% of the corresponding site measured values at two out of the three depths that were tested. Predicted velocities at the bed bear the least resemblance to the site measured values suggesting there may be a problem with the way in which boundary roughness is being incorporated into the three dimensional model.

The validation of the predicted velocities from SSIIM is not particularly encouraging, showing some significant discrepancies between the predicted and site measured velocities. Only 3.3% to 15.0% of the post scheme predicted velocities are within 50% of the corresponding site values at the locations validated with the deflectors. There are issues over the representation of boundary roughness and the modelling of turbulence in the two dimensional model that appear to require some reconsideration. As a result, SSIIM could not be relied upon to predict velocities following a rehabilitation that involves the construction of flow deflectors. In different circumstances, for example where the flow structure is less complex, SSIIM may reproduce the velocities more accurately. Other two dimensional codes, such as

Telemac2d, may predict velocities where the flow structure is complex with more accuracy. However, the results from the work carried out in this thesis indicate that a three dimensional model is to be preferred to the use of a two dimensional model for the prediction of post scheme velocities, at the design stage.

The flow modelling carried out in chapter 4 highlights the most useful application of the various packages used. The one dimensional packages are the best suited to assessing the effect of rehabilitation measures on flood levels. However, one dimensional models are not sufficient if it is necessary to predict the changes to the flow pattern arising from rehabilitation proposals. Here, two and three dimensional models are required.

One dimensional flow modelling packages are widely used in industry. If the potential benefits of two and three dimensional modelling are to be transferred to industry, the end users must become convinced of the reliability of these types of software. Studies, such as the flow verification process carried out here, are required to demonstrate the accuracy of the predictions from these packages.

Three dimensional modelling in particular suffers from the additional problem that a tailor made 3d code does not exist that is specifically aimed at river modelling. The 3d package, CFX, used in this thesis was found to be particularly time consuming in so far as the setting up of the geometry of problems was concerned. Practising engineers are unlikely to be convinced to take advantage of the additional accuracy provided by more detailed modelling until the ease with which these packages can be applied to river modelling is improved.

7.1.2 Habitat Modelling

The habitat modelling for this thesis is described in chapter 5. Since the major purpose of rehabilitating the Idle was to improve aquatic habitat, having some means to assess potential habitat improvements from alternative designs at an early stage would have been beneficial. However, the habitat modelling outlined here was performed after the rehabilitation structures had been installed.

The modelling of aquatic habitat on the River Idle carried out for this thesis uses the concepts that underlay the Instream Flow Incremental Methodology (IFIM). This is the most popular approach currently available. Here the habitat requirements of individual 'target' species of fish are described by 'preference curves', which relate habitat suitability to flow depth, velocity and the bed substrate. The IFIM technique is usually only applied to one dimensional modelling using a piece of software called PHABSIM. In this thesis the underlying methodology is applied to one and three dimensional results. Results from the habitat modelling include graphs of available areas of habitat (WUA), as well as shaded contour plots of habitat indices for the target species. Presenting contoured colour plots of the habitat indices is quite novel in habitat modelling. It allows the spatial allocation of the habitat to be seen. It is only be done with any amount of accuracy with the two and three dimensional models, which predict the flow field with much more accuracy that the one dimensional model.

The results from the habitat modelling reveal increases in available habitat for adult chub and spawning chub following the installation of the deflectors. These habitat improvements manifest in an increase in both the available area of habitat (WUA) and in the peak value of the combined habitat suitability index (the combined habitat suitability index is explained in detail in chapter 5). For roach fry the situation is the opposite with falls in both habitat measures indicating a worsening of conditions. This worsening of habitat for roach fry is unfortunate since it is already in short supply at the higher discharges modelled.

Fry require both shallow water depths and low flow velocities. It is anticipated that the installation of the deflectors would produce just these conditions in the lee of the structures, where a shelter from the main current would be produced. The results from the habitat modelling suggest this may not be the case. However, the habitat modelling for the scenarios after the deflectors were installed were carried out with the same cross sections as used in the pre scheme case. Thus, the case being modelled was that present immediately after the deflectors were installed. After the deflectors had been in place for some time it has been observed that the shape of the river bed changes (this change is also predicted in chapter 6). A pool of erosion is created at the deflector tip and deposition is seen to take place behind the deflector increasing the bed height. The increased bed height behind the deflector can reasonably be expected to give rise to lower velocities and this may produce a habitat which is far more suitable for coarse fish fry. Thus, the situation for fry may not be as poor as the results from the habitat modelling would at first suggest. More modelling with the post deflector bed forms would be required to give a more accurate indication of the changing pattern of habitat availability for all of the target species.

As far as the habitat modelling results from the one and three dimensional approaches were concerned, the results bear a good degree of similarity to each other for the pre deflector situation. For the post deflector situation, the three dimensional model predicts considerably greater increases in both the combined habitat suitability index and the Weighted Usable Area of habitat than the one dimensional model. The narrowing of the channel cross section produced by the installation of the deflectors gives rise to a plume of higher velocity which propagates downstream from the deflectors for a considerable distance. This plume of higher velocity is associated with a higher suitability value for both adult chub and spawning chub, at the majority of the discharges modelled. The plume of higher velocity is modelled in the 3d package but is absent from the 1d software where flow velocities are averaged out over larger ‘panels’ across the width of the channel. The end result is that the 1d software inevitably under predicts the beneficial effect of the deflectors on aquatic habitat. The 3d software, which predicts the flow field with more accuracy because it solves the momentum equations in all three directions, models the habitat improvement more accurately. This is a very important point, and probably the major conclusion to be drawn from this thesis. Two and three dimensional models have the potential to reliably predict flow patterns after alterations have been made to the channel. Thus, accurate predictions of the effect on habitats can be made. 1d models, which are currently widely used in industry, cannot predict altered flow patterns after changes to the watercourse and cannot be used with any confidence to predict habitat changes. Indeed the distribution of velocities across the cross section predicted by 1d models prior to any alterations must be treated with caution.

No electro-fishing surveys have been carried out since the installation of the deflectors. As a result, it is not possible to calibrate the habitat predictions against site observations at this time. This would provide a very useful piece of research to follow on from this thesis. However, it should be borne in mind that predictions of WUA from the IFIM technique are not predictions of fish biomass. Neither can they be directly correlated with fish biomass since too many complicating factors exist. This matter is discussed later in this chapter and in chapter 5.

7.1.3 Sediment modelling

The sediment modelling in this thesis was outlined in chapter 6. It consisted of two distinct parts; one dimensional modelling using ISIS Sediment and three dimensional modelling carried out by examining the changes in the pattern of bed shear stress produced by installing the deflectors.

The ISIS Sediment module predicts the changing bed elevation, due to erosion and deposition, along any modelled reach. Both the Engelund-Hansen and the Ackers and White transport equations were used. Long sections plotted through the deflector sites show downward spikes of erosion forming at the cross sections where the deflectors are installed. This matches well with site observations at the deflectors that have noted a pool of erosion forming adjacent to the deflector tips. Here the narrowing of the channel width produces faster flow and subsequently higher bed shear stresses.

The actual magnitude of the erosion and deposition that is predicted by ISIS Sediment cannot be relied upon because only a small number of discharges were modelled (not entire flow and sediment hydrographs for the period since the deflectors were installed). In addition, the incoming sediment discharge rate (to the modelled reaches) was not based on a site measure of the actual rate. Instead incoming sediment discharge rates were derived from an analysis of the sediment transport capacity of the Idle using the Brownlie equations (this data was supplied by the School of

Geography).

Despite the limitations of the ISIS Sediment results, they do at least indicate the likely change in the pattern of erosion and deposition caused by the deflectors. The results also indicate that the extra material eroded at the deflectors will be largely deposited in the zones between the deflectors. It will not be transported further downstream. This appears reasonable as the transport capacity of the river as a whole has not been increased, it has only risen at a number of very localised locations at the deflectors.

The three dimensional sediment modelling, again using CFX, is based on computations of the bed shear stress using the predicted vertical velocity profiles. In addition, eroded areas of bed (for specified particle diameters) are computed using the concept of a critical bed shear stress from the Shield's diagram. Results show increases in the calculated bed shear stress at the deflector tip and extending downstream. The eroded area of bed is also associated with the zone of increased shear stress at the deflector tip. This matches site observations as outlined above. The increase in bed shear stress, due to the deflectors, is as high as 136%. The increase in eroded area of bed can be several thousand %. Deposition of material is not modelled directly using the results from CFX. However, areas where deposition is likely can be implied from locations where low shear stresses are produced. This is evident in the lee of each of the deflectors and, again, this matches site observations well.

Prior to the installation of the deflectors, the flow in the Idle was very uniform and was insufficient to transport away the large volumes of wind blown sediment that is thought to derive from adjoining agricultural land. The installation of the deflectors appears to have been successful in creating small zones of higher bed shear stress where the bed sediment is eroded to form a pool. The eroded material is deposited in the region between the deflectors. A zone of deposition is also created in the lee of each of the deflectors. All of these effects have been observed in the results from the one and three dimensional modelling outlined. A report is pending from the School of Geography at The University of Nottingham outlining in detail the changes that have occurred in the Idle, at each deflector, since the rehabilitation scheme. This report is expected to be published this summer. It would be very useful to compare the predicted changes in bed shear stress and eroded areas of bed, for each deflector site, outlined here with the actual changes that took place at each site (Downs 1999). This would give a good indication of the success of the modelling technique employed here. A qualitative comparison of the results from the sediment modelling exercise and initial findings from the post scheme surveys of the deflector sites, carried out by the by the School of Geography, is included in chapter 6. It is not possible to infer a great amount from this comparison, but the major features that are evident on site (the development of a scour pool and deposition above and below the deflector) are predicted in the models.

7.2. Future Work on the River Idle study

- There is a large volume of additional work that could be completed to extend the scope of this study. The most important would be to model the same deflector sites with the altered shape of the channel cross sections that exist several months after the rehabilitation. Here, the pool of erosion at the tip of each deflector would be present, as well as the deposited material in the deflectors lee. The altered bedform would have great significance for species habitat (as discussed above). Not only would the depths alter, but the velocities around the deflectors would also be very different. The available habitat and suitability indices for the target species would both be substantially altered and this would have great consequences for the perceived habitat improvement. In addition, the altered bedform would substantially effect the bed shear stresses meaning that the pattern of erosion and deposition in the various sites would be altered. The increased flow depth at the deflector would eventually lead to lower velocities and reduced bed shear stress. A point would come where the pool was fully formed and no more erosion would take place. It would be possible to see if this condition could be predicted by modelling successively deeper scour holes until the shear stress became insufficient to erode the bed material.

- It would be useful to complete extra work to test, and try to improve the accuracy of, the two and three dimensional models. This would require many more two dimensional site measurements of velocity to be taken. The advantage of this is that it would allow better calibration of the model predictions. The site measurements should extend well upstream and downstream of the deflectors so that the accuracy of much more of the predicted flow field can be examined.

- The results from SSIIM calibrate particularly badly with site measurements of velocity. Extra work is needed to investigate the cause of this discrepancy in values. There are issues that specifically need to be addressed with regard to the representation of boundary roughness and the modelling of turbulence.

- It is also necessary to examine the way in which boundary roughness is being incorporated into the three dimensional model. Boundary roughness is modelled by varying parameters which specify parameters which relate to the logarithmic portion of the boundary layer. The present method of relating the particle grain size to the boundary layer coefficients needs to be examined.

- The validation process itself also needs to be looked at. At present, validation is carried out using a FORTRAN program written by the author which takes a weighted mean of the four spatially nearest predicted velocities to each site velocity. It may be better to examine ways to import the contoured predicted velocities into AutoCAD so that a value can be read off at the locations where site velocities have been measured.

- A number of factors could be explored to improve the habitat modelling carried out. These could include modelling more deflector sites and a greater range of discharges. In addition, the habitat for more target species and more life stages could be simulated. This would produce a larger volume of results so the effect of the rehabilitation would be assessed in greater depth. However, it is likely that the volume of results would become overwhelming and, thus, hard to draw overall conclusions from. If habitat improves for one species but declines for another it is difficult to say whether the situation has improved overall or worsened.

- Further possibilities for habitat modelling of the Idle rehabilitation include using another habitat model (such as RCHARC) and comparing and contrasting the results from the two models. Fish surveys need to be carried out to accurately quantify the improvement in fish biomass and see if there is a correlation with the habitat predictions made here.

- As far as the sediment modelling is concerned, probably the major piece of work that needs completing is to verify the results from the two modelling approaches. The report that is due from the School of Geography may go some way to doing this. At this stage only a small amount of qualitative validation has been possible, using some of the initial findings of the effect of the deflectors. This initial validation appears to be encouraging.

- In order to validate the predictions from ISIS sediment, it would be necessary to run the model with a site measured sediment hydrograph (not the synthetic one used here) and the associated flow hydrograph. Predicted changes in bed elevation over the period could then be compared with recorded changes on site. This would enable an assessment of which set of sediment transport equations (Engelund Hansen or Ackers and White) best reproduces the site measurements in this case.

- Validating the results from the three dimensional sediment modelling would be a lot harder since the method uses only a single value of discharge at a time (so it cannot recreate a continuous hydrograph) and it does not make direct predictions of erosion depths. The present results can only be validated in a qualitative sense by comparing the predictions for eroded areas against the areas in which scour pools were seen to form.

- The three dimensional modelling technique used could be improved by incorporating the effect of the existence of a range of particle diameters, and associated sediment armouring, into the computation of the critical shear stress for erosion of the bed. In addition, some form of particle tracking of individual grains could be carried out. Tracking the paths of sediment grains may allow determination of likely amounts of material that will be caught in the recirculation behind the deflectors and thus become deposited in this region. At present, deposition areas are implied where zones of low shear stress exist. This implies that sediment is transported into these areas, but this may not be the case. Tracking large numbers of particle grains would give an indication that sediment will be transported into these areas.

- A further possibility with the three dimensional modelling would be to examine the effect of using different percentages of the depth in the calculation of the bed shear stress. The current method employed by the SHEAR program uses only the bottom 20% of the velocity profile. Several researchers have suggested using larger amounts of the velocity profile in the calculation. Another possibility would be to look at the effect of using the predicted turbulence near the boundary to calculate the wall shear stress, instead of the velocity profile. Some site measurements of bed shear stresses would be needed to validate the various results from these methods.

- Further improvements to the three dimensional modelling of sediment movements would be to include some form of slope stability analysis. This would allow the mass failure of parts of the bank to be modelled. In addition, the erosion of sediment on the banks should be considered differently to particles on the bed because the part of the self weight of the material on a slope helps in dislodging it from its position.

- In the study of the transport of material, flushing flows are of great importance. These are relatively high discharges that occur infrequently and have the ability to erode and transport particles with larger grain sizes. These pulses of larger discharge are important in analysing the sediment transport in a river. However, since only single values of discharge can be modelled at a time on CFX, and no sediment hydrograph is available for the 1d modelling, the significance of the flushing flows has not been modelled.

7.3. Recommendations for Further Research within the related area

7.3.1 Rehabilitation

- The rehabilitation of rivers is a relatively new concept. The eco-hydraulics community as a whole is still on a learning curve in terms of deciding what can and cannot be achieved by each rehabilitation design. In some cases it may be possible to make very significant habitat improvements, while in other cases the requirements for stability or flood defence may be more pressing. Even where the requirements of a scheme can be clearly defined, modelling techniques are still not sufficiently developed to be certain that a specific design will be completely stable or that it maximises the potential for habitat improvement. Considerably more research is required in these areas, some of which is outlined in the sections below, before we can reach this stage. One step that ought to be completed is to carry out pre- and post-project appraisals (in terms of stability and habitat improvement) on all rehabilitation schemes, so that the effect of each project can be defined and lessons learnt from any projects that created unfavourable results. By doing this it should become possible to eliminate poor designs and, in future, concentrate on those that created the most stability and/or habitat benefits.

- Those making management decisions with regard to rehabilitation schemes can attempt to assist the process of design by attempting to understand the limitations and possibilities of each design and, with these in mind, to define specific goals. These may include maximising bio-diversity, targeting the design at habitat improvement for a single species, ensuring stability, creating a natural looking channel or maintaining flood defence. This process can be encouraged by ensuring there is input from an ecologist, engineer and geomorphologist at an early stage and before a design even reaches the drawing board.

7.3.2 Flow Modelling

- The drawbacks of current 3D models have been commented on in several places in this thesis. A tailored 3D code is required specifically for open channel modelling. The code should automatically include all the important parameters in its calculation of depths and velocities, and should not require the user to decide which parameters need to be included, and which have little or no effect. Also the end user should not need to have a detailed knowledge of turbulence modelling or CFD in general. The code would also need to be calibrated against a large number of real river measurements to ensure the predicted velocities are accurate. The CFX package could be developed to the point where it fulfils many of the requirements listed. This package would need to be improved so that it is able to calculate the position of the free surface accurately.

- The SSIIM model needs to be tested on more rehabilitation sites to determine its accuracy. The study here is not sufficiently in depth to comment on this in too much detail. It is possible that some of the inaccuracies found in SSIIM here are generic to two dimensional models. Alternatively, it may be a problem with the application of SSIIM to 'real' river channels. A detailed, specific investigation would be required in order to comment in detail on the source of the inaccuracies. Two dimensional models may be a cheaper, less complicated alternative to the use of a full three dimensional model. They should not be overlooked in the search for a tool capable of accurately predicting post rehabilitation depths and velocities.

- The required amount of detail needs to be decided upon for modelling the different components of a rehabilitation design. A one dimensional model is certainly sufficient for analysing flood levels, and it has uses in sediment modelling. However for detailed modelling of erosion where the flow is clearly three dimensional, a more complex model may have to be used.

- The report into the floods of Easter 1998 pointed out that shortfalls still exist with one dimensional models (Independent Review Team, 1998). It is important to ensure that predictions from any model are accurately validated against recorded flood data. More research is still needed to improve and develop the performance of existing one dimensional models.

7.3.3 Habitat Modelling

- Habitat modelling is still a relatively new technology and a considerable amount of research is still required to develop the field. Research in this area is often difficult because the objects of the study, the species present, are hard to examine without disturbing them. However, more knowledge is required of: biological interactions and the effects of predation, prey availability and competition on fish biomass.

- In addition, more consideration needs to be given to microinvertebrates to define their habitat requirements and to make sure that they are being accommodated in schemes which seek to improve the aquatic environment

- A further need is to assess the effect of the habitat mosaic, whose potential importance is only just beginning to be realised. Two and three dimensional flow models present a great opportunity for the eco-hydraulics community. The flow field can now be predicted in detail. However, this great potential benefit is limited by the fact that habitat requirements are still defined in IFIM by single values of depth and flow velocity. Much more research is required in this field by ecologists so that habitat requirements, in terms of the whole of the flow field, can be defined in detail. If this information were available, then habitat improvements could be modelled with much more confidence using two and three dimensional models.

- Habitat requirements of fish communities (those found to commonly co-exist in the same conditions) also need to be assessed, rather than individual species. In this way, habitat models could deal with the needs of the whole range of species present rather than having to select 'target' species and exclude others from the study.

- Research is also needed into the use of nose velocities in assessing habitat. In the analysis presented here the depth averaged velocity, at each point, was used as the input to the habitat model. In reality, fish can move vertically up and down, to a certain extent, to select the velocity (nose velocity) that they find most suitable. However, some fish are either bottom or surface feeders so their movement is more limited. The extent to which some fish are free to move vertically, whilst others are to a lesser amount, needs establishing so that the correct velocity, or range of velocities, can be extracted from the results of the hydraulic model and used as input to the habitat model.

- Finally, a criteria needs to be developed to govern the transferability of species preference curves between watercourses. At present, it is at the modeller's discretion whether a different watercourse is sufficiently 'similar' that it is permissible to use species preference curves from it, on the river that is under consideration.

7.3.4 Sediment Modelling

- There is a large quantity of literature available on sediment transport modelling. In addition, detailed sediment transport modelling in two or three dimensions can be extremely complex. The sediment transport modelling in this thesis that uses the results from the CFX model is based on an analysis of bed shear stress only. It is relatively simple and has certain limitations and assumptions. However, it is a first step towards a model that could predict: sediment transport capacities, bank erosion, erosion and deposition on the bed and the mass failure of the side slopes of the channel when the slope exceeded a critical value. This type of model would be extremely complex but would enable a full stability analysis of any rehabilitation design to be tested. In addition, the long term evolution of the channel could be predicted.

7.4. Comments with regard to the objectives of the thesis

To conclude this thesis, consideration will now be given to the success of the modelling work undertaken in achieving the objectives set out in chapters 1 and 2. The work carried out has highlighted the hydroinformatic possibilities in current state-of-the-art modelling packages with respect to river rehabilitation schemes. The useful data that can be obtained from each package has been demonstrated, together with what can be interpreted from the results obtained.

In chapter four, the one dimensional packages ISIS and HEC-RAS were used to simulate the increase in water level arising from the installation of the deflectors. In the same chapter the detail of the flow structure, in the vicinity of a number of deflectors and at several different discharges, was modelled using SSIIM and CFX.

Although the validation of the SSIIM predicted velocities was poor, compared to site measurements, the work undertaken highlights the useful data that can be obtained from the different packages. One dimensional models are useful in flood prediction calculations. Two and three dimensional models offer the ability to predict the spatial detail of the flow structure in more detail.

The importance of the increased accuracy that can be obtained in predicting the spatial detail of flow structure using a three dimensional model is highlighted in chapter five. In this chapter, comparison of habitat predictions from a one and three dimensional model show significant differences. The greater accuracy and detail obtained in habitat predictions from a three dimensional model demonstrate considerable advantages in the use of this type of approach. The advantages of using a three dimensional model are further illustrated in chapter 6 where the spatial detail of erosion and deposition patterns around flow deflectors is successfully obtained from the three dimensional results. In addition, by carrying out habitat and morphology predictions at the same time it is demonstrated that it is possible to make some implications about the quality and quantity of aquatic habitat that will be available in future. Thus, it is possible to make predictions of the longer term effect of rehabilitation on the channel and on the aquatic habitat it provides.

The importance of a morphological assessment of the condition of the channel prior to rehabilitation is also discussed in detail in chapter 6. In effect, the morphological assessment of a river can be said to provide the larger picture of the river in which the rehabilitation is to be set. It is important that input from both a gemorphologist and engineer is sought on any rehabilitation. The geomorphologist can advise on the governing processes in the channel and, in effect, the context within which the rehabilitation is set. The engineer tends to be more focused on the detail of modelling at the rehabilitation site. Thus, the twin approaches are complimentary and equally important.

Overall, the useful data that can be obtained from a variety of open channel modelling packages has been highlighted. The packages have been used to make predictions in the three key areas for any rehabilitation scheme of: flood defence, aquatic habitat and channel stability. By assessing flood defence, habitat and stability considerations at the same time, a unified approach has been successfully applied taking into account all the key areas. In addition, by carrying out these analysis at the same time it is possible to use predictions of morphological change to imply the effect of rehabilitation on habitat availability over extended timescales.

In chapter one it was stated that the tools were now increasingly becoming available to make reliable predictions with regard to rehabilitation schemes, and that guesswork could be replaced by accurate prediction. A number of these tools have been demonstrated in this thesis, highlighting that it is now possible to make accurate predictions with regard to all the key areas relevant to rehabilitating or restoring a river. In addition, the question of the required level of accuracy (1d, 2d or 3d) for modelling rehabilitation schemes was also raised in chapter one. The advantages of a three dimensional approach have been demonstrated in the case of the River Idle rehabilitation scheme. The rehabilitation took the form of the installation of flow deflectors. These had a pronounced effect on the localised flow pattern leading to improvements in habitat and in morphological change. In this case a three dimensional model is inevitably of more use as it is able to predict flow patters in great detail. Therefore, for this type of rehabilitation a three dimensional modelling approach shows great benefits. However, for a different style of restoration (for example direct reinstatement of a granular substrate) a three dimensional approach may have much less success. This area requires much more research, and it still remains to be determined which type of modelling approach is appropriate for each style of rehabilitation.

With specific regard to the effect of the River Idle restoration, the results in chapter four show that the likely effect of the installation of the deflectors is a rise in flood level of only 0.13m (NRA, 1995). With normal levels of maintenance this would be reduced to a rise of only 0.05m. This is significant as one of the requirements of the rehabilitation was not to significantly worsen the risk of flooding, as discussed in chapter two.

The fact that the Idle is currently a relatively poor fishery is also discussed in chapter two. The desire to increase the fisheries value of the channel was one of the aims of the recent restoration. The results of the habitat modelling in chapter five show that habitat is improved significantly for adult and spawning chub by the installation of the deflectors. This habitat improvement is associated with a plume of higher velocity flow which is produced by the narrowing of the channel width at the deflectors. Unfortunately, available habitat appears to be worsened for roach fry. However, as a bank of silt builds up behind the deflectors over time, habitat for this species life stage can be expected to improve. This is because good quality habitat for roach fry is associated with low velocities and shallow depths.

The morphological survey of the Idle, discussed in chapter two, highlighted a very low bank full stream power. This meant that the Idle formed a depositional environment as was evidenced by the large amounts of sand accumulated on the river bed. The rehabilitation proposals were intended to prompt the natural flow processes of the river to develop some morphological diversity of the bed. The subsequent sediment modelling of chapter six shows that a zone of increased shear stress is produced adjacent to the deflector tips. The shear stress produced is sufficient to cause erosion of the sand substrate. This has led to the development of a scour pool. In addition, the eroded sediment is deposited in the lee of the deflector, as implied by areas of reduced shear stress in the sediment modelling results. Thus, the deflectors do appear to have been successful in working with the natural flow processes in the river to create some morphological diversity within the channel. As indicated, the observed effect is indicated well by the results of the sediment modelling exercise.

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Appendix A

Photographs of deflectors 6a, 3f and 3c in the River Idle

Plate 1. Deflector 6a immediately after installation, January 1996. Looking downstream. Evidence of flow separation can be seen at the deflector tip.

Plate 2. Deflector 6a, October 1996. Looking downstream. The deflector has become overgrown and forms a permanent feature of the riverbank.

Plate 3. Deflector 3f, October 1996, looking downstream. Growth of vegetation on the deflector tops is evident.

Plate 4. Deflector 3c, October 1996. Looking upstream. Again, vegetation is beginning to take hold on top of the deflectors.

Publications Associated with this thesis

Downs P. W., Wright N. G., Swindale N. R., Skinner K. S., 'Modelling Detailed Hydraulic and Morphological Change Following Installation of Flow Deflectors in the River Idle, Nottinghamshire, UK', Proceedings of the International Symposium on Ecohydraulics, Salt Lake City, 1999.

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